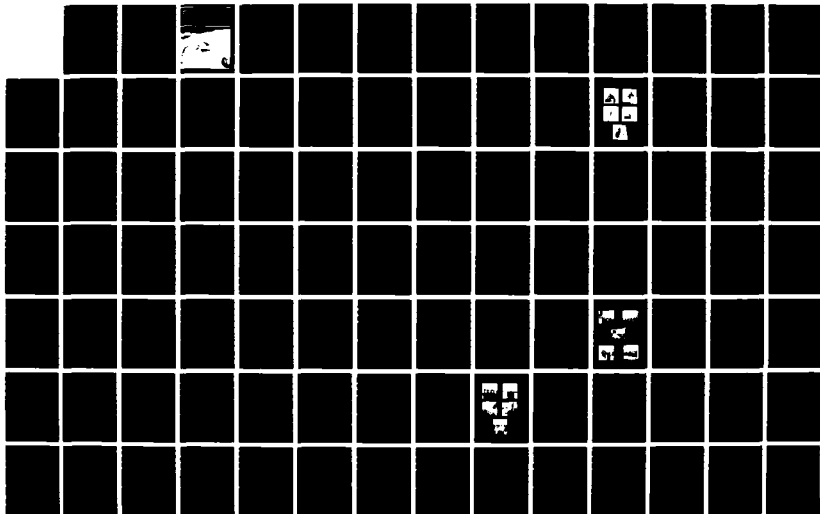


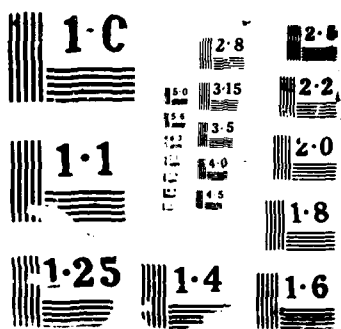
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COMPLETION REPORT ON THE CORPS OF ENGINEERS STRUCTURAL 1/2  
ENGINEERING CONFER. (U) OFFICE OF THE CHIEF OF  
ENGINEERS (ARMY) WASHINGTON DC L GUTHRIE ET AL. MAR 86

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US Army Corps  
of Engineers

Office of the Chief  
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# Completion Report on the Corps of Engineers Structural Engineering Conference

Portland Marriott  
1401 SW Front Ave., Portland, Oregon  
June 23-28, 1985

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)  This is a document of material assembled as a completion report of the Corps of Engineers Structural Engineering Conference held in Portland, Ore., 23-28 June 1985. Presented under this cover are the agenda of the conference, abstracts of the technical presentations, a description of exhibits displayed, a list of attendees, and an evaluation of the conference by the participants.  (Continued)		

## 20. ABSTRACT (Continued).

Training and demonstration sessions were an integral part of this conference. The training sessions were conducted to review recently published and draft design guides and manuals. The training was aimed at providing instructions on the use of and receiving constructive comments on these instructive materials. The demonstrations, presented as introductions to computer programs developed under the Computer-Aided Structural Engineering Project, created additional interest in CASE.

Sponsored by the Engineering Division, Engineering and Construction Directorate of the Office, Chief of Engineers, US Army (OCE), the conference provided for the exchange of structural engineering experience in project design and construction, methods of structural analysis, and remedial measures. A further purpose of the conference was for review of OCE-sponsored structural engineering research and computer-aided design programs provided by the training and demonstration sessions.

Represented at this conference were 40 Corps field offices, OCE, the Waterways Experiment Station, the Coastal Engineering Research Lab, 9 non-Corps offices, and 3 universities, totalling 202 in attendance.

## PREFACE

The Engineering Division, Engineering and Construction Directorate (formerly Civil Works Directorate), of the Office, Chief of Engineers, US Army (OCE), sponsored a Corps of Engineers Structural Engineering Conference 23-28 June 1985 in Portland, Ore. The purposes of this conference were to provide for exchange of structural engineering experience in project design and construction, methods of structural analysis and remedial measures, and for review of OCE-sponsored structural engineering research and computer-aided design programs.

The conference was attended by 187 Corps engineers from 40 Corps field offices, OCE, the Construction Engineering Research Laboratory, and the Waterways Experiment Station (WES), and 14 engineers representing 9 non-Corps offices, including the Federal Energy Regulatory Commission, Tennessee Valley Authority, US Department of Agriculture Soil Conservation Service, US Bureau of Reclamation, Oklahoma State University, University of Florida, and University of Portland.

The five-day conference included state-of-the-practice sessions, state-of-the-art sessions, training and demonstration sessions, and a field trip. Included was an afternoon of concurrent sessions in which "snapshot" demonstrations were given on computer programs developed under the Computer-Aided Structural Engineering (CASE) Project. Training sessions on recently released ETL's and on-going work of interest to the field were also conducted.

The conference was planned, organized, and conducted by an OCE-appointed steering committee composed of the following Corps personnel:

Mr. Lucian Guthrie, Structural Engineering Section, OCE

Mr. Seichi Konno and Mr. David Illias, Portland District

Mr. Paul K. Senter, Automation Technology Center (ATC), WES

Mr. Guthrie served as OCE point of contact. The Portland District had responsibility for coordinating local arrangements. Mr. Konno was in charge of the Portland District's coordinating efforts. Mr. Senter coordinated the CASE demonstration sessions and he and Dr. N. Radhakrishnan, WES, ATC, assisted in the editing of this document. The photographs for the cover and those placed throughout the report were supplied by Mr. John White of the Sacramento District and Mr. Bill Johnson, Portland District Photography Laboratory. Publications and Graphic Arts Division, WES, Editor Gilda Shurden assembled

and provided final editing of the material for this document, and Editorial Assistant Frances Williams coordinated the layout design for the cover and text of this report.

COL Allen F. Grum, USA, was Director and Dr. Robert W. Whalin was Technical Director at the time of publication.

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## AGENDA

US ARMY CORPS OF ENGINEERS  
STRUCTURAL ENGINEERING CORPS-WIDE CONFERENCE

23-28 June 1985

Portland, Oregon

AGENDA

SUNDAY - 23 June 1985

5:00 p.m.      Registration  
          to  
8:00 p.m.

MONDAY - 24 June 1985

7:00 a.m.      Registration

8:00 a.m.      SESSION 1 - Chairman, Mr. Ivar Paavola, Acting Chief,  
                 Structural Engineering Section, OCE

8:05 a.m.      Welcoming Remarks - Col. Robert L. Friedenwald,  
                 Commander, NPP

8:10 a.m.      Announcements - Mr. Seichi Konno, Chief, Structural  
                 & Architectural Design Section, NPP

8:15 a.m.      Conference Overview - Mr. Robert J. Smith, Chief,  
                 Structures Branch, E&C Directorate, OCE

8:20 a.m.      Keynote Address - Mr. William N. McCormick, Chief,  
                 Engineering Division, E&C Directorate, OCE

8:50 a.m.      Mount St. Helens-Spirit Lake Outlet Tunnel - Mr. Donald R.  
                 Chambers, Structural & Architectural Design Section, NPP

9:20 a.m.      BREAK

CONCURRENT SESSIONS

9:40 a.m.      SESSION 2A - Chairman, Mr. Victor M. Agostinelli, Chief,  
                 Structural Section, LMVD

9:45 a.m.      L&D 26R, Cofferdam Testing Program - Mr. Richard J. Flauaus,  
                 Structural Engineering Section, LMS & Mr. Reed L. Mosher,  
                 Engineering Applications Group, WES

10:30 a.m.      Three-Dimensional Finite-Element Analysis of a Cellular  
                 Cofferdam - Mr. Reed L. Mosher, Engineering Applications  
                 Group, WES

MONDAY - 24 June 1985

CONCURRENT SESSIONS

- 9:40 a.m. SESSION 2B - Chairman, Mr. Donald L. Bergner, General Engineering Section, SPD
- 9:45 a.m. Munitions Storage Magazines, Structural Failure and Evaluation of Steel Arch "Standard Design" - Mr. Kirk M. Price, Reservoir Structures Sections, MRK
- 10:30 a.m. CTABS80 Usage in Building Design - Mr. Dennis E. Bellet, Military Design Section B, SPK
- 11:00 a.m. STRETCH BREAK

CONCURRENT SESSIONS

- 11:10 a.m. SESSION 3A - Chairman, Mr. Victor M. Agostinelli, Structural Section, LMVD
- 11:15 a.m. Crater Lake Hydroelectric Project - Mr. Joseph B. Leeak, Structures Section, NPA
- 11:10 a.m. SESSION 3B - Chairman, Mr. Donald L. Bergner, General Engineering Section, SPD
- 11:15 a.m. Geodesic Dome Structure, Design and Construction - Messrs. Pete Lam & Jim Nott, Structures Section NPA
- 11:45 a.m. LUNCH
- 1:00 p.m. SESSION 4 - Chairman, Mr. Donald R. Dressler, Structural Engineering Section, OCE
- 1:05 p.m. Status Report on the Computer-Aided Structural Engineering (CASE) Project - Dr. N. Radhakrishnan, Chief, Automation Technology Center, WES
- 1:35 p.m. Computer-Aided Design and Drafting Studies - Mr. John E. Naeger, Design Branch, LMS. Demonstration of Computer-Aided Design and Drafting System Capabilities - Robert Holt and Roger Hoell, St. Louis District
- 2:00 p.m. Microcomputers in Structural Engineering - Mr. Bill A. Price, Chief, Engineering Applications Group, WES
- 2:25 p.m. BREAK
- SESSION 5
- 2:45 p.m. Concurrent 25-Minute Introductory Sessions for CASE Programs  
to  
4:40 p.m. (See page 14)

MONDAY - 24 June 1985

4:40 p.m. ADJOURN  
5:30 p.m. ICE BREAKER  
to  
6:30 p.m.

TUESDAY - 25 June 1985

8:00 a.m. SESSION 6 - Chairman, Mr. Lucian G. Guthrie, Structural Engineering Section, OCE

8:05 a.m. The Engineer's Responsibility for Structural Integrity and Safety - General Robert D. Bay (USA, Ret), Black & Veatch E/A

8:35 a.m. Old River Control Auxiliary Structure, Posttensioning System - Messrs. Thomas Hassenboehler & Alan Schulz, Structural Design Section, LMN

9:20 a.m. BREAK

CONCURRENT SESSIONS

9:40 a.m. SESSION 7A - Chairman, Mr. James G. Lewis, Chief, Structural Section, SWT

9:45 a.m. Prototype Dynamic Testing of the Richard B. Russell Concrete Gravity Dam - Mr. Vince P. Chiarito, Structures Laboratory, WES

10:30 a.m. Three-D Dynamic Analysis of Englebright Arch Dam - Mr. John W. White, Chief, Civil Design Section A, SPK

9:40 a.m. SESSION 7B - Chairman, Mr. George Henson, Chief, Structural Section, SWT

9:45 a.m. Tactical Equipment Shops, Design and Layout Study - Mr. Gary R. Close, Structural Section, SAS

10:30 a.m. Investigation of Structural Damage of Tactical Equipment Shops, Fort Stewart - Mr. W. T. Cheung, SAS

11:00 a.m. STRETCH BREAK

11:10 a.m. SESSION 8A - Chairman, Mr. James G. Lewis, Chief, Structural Section, SAD

11:15 a.m. Simplified Load Factors for ETL 1110-2-265, Strength Design for Reinforced Concrete Hydraulic Structures - Mr. C. C. Hamby, Structures & Civil Section, LMK

TUESDAY - 25 June 1985

11:10 a.m. SESSION 8B

11:15 a.m. Military Project Design - Messrs. Allan G. Wesley &  
Larry Cozine, ORL

11:45 a.m. LUNCH

CONCURRENT SESSIONS

1:00 p.m. SESSION 9A - Chairman, Mr. Leland D. Anderson, Structural  
Section, SAD

1:05 p.m. Post-and-Panel-Type Retaining Wall - Mr. Bryon K. McClellan,  
Chief, Structural Section, ORL

1:35 p.m. Structural and Architectural Design Features of Flood Walls -  
Mr. Jorge A. Romero, Structural Design Section, LMN

1:00 p.m. SESSION 9B - Chairman, Mr. Tom Wright, Reservoir Structures  
Section, MRK

1:05 p.m. Roller-Compacted Concrete for Elk Creek Dam - Mr. Dennis R.  
Hopman, Chief, Concrete Control Section, NPP

1:35 p.m. Converting from In-House to A/E Contracting - Mr. John C.  
Kliethermes, Design Branch, NCS

2:05 p.m. BREAK

SESSION 10

2:25 p.m. Concurrent 40-Minute Training Sessions  
to (See page 14)  
5:20 p.m.

WEDNESDAY - 26 June 1985

8:00 a.m. SESSION 11 - Chairman, Mr. Melvin J. Setvin, Chief,  
Technical Engineering Branch, NPD

8:05 a.m. "Fast Track" Design/Construction of the Central Command HQ,  
MacDill AFB - Mr. Thomas L. Fultz, General Structures Section,  
SAM

8:35 a.m. Bonneville Navigation Lock Structural Design - Mr. Norman P.  
Tolonen, Coordinator, Columbia River Projects, NPP, Mr. Jerry  
Maurseth, Structural/Architectural Design Section, NPP

9:20 a.m. BREAK

WEDNESDAY - 26 June 1985

CONCURRENT SESSIONS

9:40 a.m. SESSION 12A - Chairman, Mr. Victor M. Agostinelli, Structural Section, LMVD

9:45 a.m. L&D 26R First Stage Dam, Design, and Construction, Case History - Mr. Thomas J. Mudd, Structural Engineering Section, LMS

10:30 a.m. L&D 26R Lock, Design, and Construction Sequence - Mr. Roger Hoell, Structural Engineering Section, LMS

9:40 a.m. SESSION 12B - Chairman, Mr. Ron Lein, Chief, Structural Section, HND

9:45 a.m. Integrated Structural Engineering Support for the FEMA Key Worker Blast Shelter Program - Mr. Paul M. LaHoud, Structural Section, HND

10:15 a.m. Dynamic Soil-Structure Interaction Effects on and Reinforcement Details for Blast Shelter Design - Dr. Sam A. Kiger & Mr. Stanley C. Woodson, Structures Laboratory, WES

11:00 a.m. STRETCH BREAK

11:10 a.m. SESSION 13A - Chairman, Mr. Victor M. Agostinelli, Structural Section, LMVD

11:15 a.m. Concrete Channels, Drainage and Freeze Protection System - Mr. John H. Plump, Chief, Design Engineering Section, NCS

11:10 a.m. SESSION 13B - Chairman, Mr. Ron Lein, Chief, Structural Section, HND

11:15 a.m. Restoration of Building 3001, Tinker AFB - Mr. Reggie Kikugawa, Structural Section, SWT

11:45 a.m. BREAK

SESSION 14

12:15 p.m. Field Trip - NPP  
to The Bonneville Lock and Dam Project  
5:00 p.m.

5:30 p.m. Salmon Bake

THURSDAY - 27 June 1985

8:00 a.m. SESSION 15 - Chairman, Mr. John L. Johnson, Asst. Chief, Engineering Division, MRD

THURSDAY - 27 June 1985

- 8:05 a.m. Problems with Long-Term Concrete Deflections in a Hot, Arid Climate - Mr. A. O. Werner, Chief, Structures Section, MED
- 8:35 a.m. Unique Factors Influencing the Design and Construction of the Ft. Campbell Hospital - Mr. Daniel L. Parrott, Project Engineering & Configuration Section, SAM
- 9:05 a.m. BREAK
- CONCURRENT SESSIONS
- 9:25 a.m. SESSION 16A - Chairman, Mr. Dave Raisanen, Hydroelectric Design Center, NPD
- 9:30 a.m. Thermal and Stress Analysis for Longitudinal Joints for 3 Gorges Dam - Mr. David A. Dollar, Analytical Design Section, USBR
- 10:00 a.m. Savings through Engineering on the Downstream Guidewall at L&D 26R - Mr. John Jaeger, Structural Engineering Section, LMS
- 9:25 a.m. SESSION 16B - Chairman, Mr. William D. Churchill, Chief, Architectural & Structural Section, MRD
- 9:30 a.m. Underground Munitions Storage Facilities Study - Dr. James D. Prendergast, Engineering & Materials Division, CERL
- 10:00 a.m. Underground Munitions Storage Complex, Design - Mr. William H. Gaube, Hardened Structures Section, MRO
- 10:30 a.m. STRETCH BREAK
- 10:40 a.m. SESSION 17A - Chairman, Mr. Ervell A. Staab, Architectural & Structural Section, MRD
- 10:45 a.m. A Comprehensive Approach to Dam Safety - Mr. James R. Jordan, Navigation & Water Resources Structures Section, SAM
- 11:15 a.m. Norfolk Dam Stress Analysis - Mr. David A. Hill, Structural Section, SWL
- 10:40 a.m. SESSION 17B - Chairman, Mr. Herman Gray, Chief, Design Branch, ORN
- 10:45 a.m. Structural Behavior of Miter Gates - Mr. Joseph P. Hartman, Structural Section, SWD
- 11:15 a.m. Structural Behavior of Alternate Configurations of Miter Gates - Mr. Michael D. Nelson, Civil Projects Management Section, NPS

THURSDAY - 27 June 1985

11:45 a.m. LUNCH

CONCURRENT SESSIONS

1:00 p.m. SESSION 18A - Chairman, Mr. Stacey C. Anastos, Structural Section, NAD

1:05 p.m. Cerrillos Dam Outlet Works, Design, and Construction, Puerto Rico - Mr. Ray Navidi, Chief, Structures Section, SAJ

1:35 p.m. Mud Mountain Dam Intake Tower Analysis - Mr. Paul C. Noyes, Structural Design Section, NPS

1:00 p.m. SESSION 18B - Chairman, Mr. Dave Ross, Technical Engineering Branch, NPD

1:05 p.m. Concrete Reactivity Problems and Remedial Measures at TVA Projects - Mr. Harold C. Buttrey, Inspection & Maintenance Group, TVA

1:35 p.m. Concrete Reactivity Problems and Remedial Measures at Center Hill Dam - Mr. Ken Hull, Civil & Structural Section, ORN

2:05 p.m. BREAK

2:25 p.m. SESSION 19A - Chairman, Mr. Bobby B. Felder, Chief, Navigation & Water Resources Structures, SAM

2:30 p.m. Caisson Foundation Design and Construction at Savannah River Plant - Mr. Kirti S. Joshi, Structural Section, SAS

3:00 p.m. L&D 25 Guidewall Repairs and Stabilization - Mr. Thomas Ruf and Thomas J. Leicht, Structural Engineering Section, LMS

2:25 p.m. SESSION 19B - Chairman, Mr. John J. Speaker, Chief, Design Branch, ORL

2:30 p.m. Major Rehabilitation of Bourne and Sagamore Highway Bridges - Mr. David R. Descoteaux, General Engineering Section, NED

3:00 p.m. Corrosion Protection of Projects in Louisville District - Mr. Ralph B. Snowberger, Structural Section, ORL

3:30 p.m. STRETCH BREAK

SESSION 20

3:40 p.m. Concurrent 40-Minute Training Sessions  
to (See page 15)

5:05 p.m.

FRIDAY - 28 June 1985

8:00 a.m. SESSION 21A - Chairman, Mr. Joe Jacobazzi, Technical Branch, NCD

8:05 a.m. Illinois Waterway, Lockport, and Brandon Road Locks 1984 Rehabilitation - Mr. Denny A. Lundberg, Project Management Section, NCR

8:35 a.m. Investigation and Repair of John Day Navigation Lock Downstream Lift Gate - Mr. William Wheeler, Structural/Architectural Design Section, NPP

8:00 a.m. SESSION 21B - Chairman, Mr. Bill M. Gray, Chief, Civil & Structural Section, ORN

8:05 a.m. Concrete Floating Breakwaters - Mr. George G. England, Structural Design Section, NPS

8:35 a.m. Fisherman's Wharf Breakwater Structural Design - Mr. Gary W. Sjelin, Design Branch, SPL

9:05 a.m. BREAK

9:25 a.m. SESSION 22A - Chairman, Mr. George Gibson, Structural Engineering Section, OCE

9:30 a.m. Miter Gate Rehabilitation - Mr. George G. England, Structural Design Section, NPS

10:00 a.m. Rehabilitation of Vertical Lift Gates, Emsworth Dams - Mr. Eugene A. Ardine, Structural Section, ORP

9:25 a.m. SESSION 22B - Chairman, Mr. M. K. Lee, Structural Engineering Section, OCE

9:30 a.m. Silica Fume Concrete Repair of Kinzua Dam Stilling Basin - Mr. Anton Krysa, Structural Section, ORP

10:00 a.m. Savannah Harbor Tide Gates, Structural Problems and Repairs - Mr. John W. Hager, Structural Section, SAS

10:30 a.m. STRETCH BREAK

10:40 a.m. SESSION 23 - Chairman, Mr. Robert J. Smith, Chief, Structural Branch, OCE

10:45 a.m. PANEL DISCUSSION

11:30 a.m. CONCLUDING REMARKS

11:45 a.m. ADJOURN

### CASE - Introductory Sessions

Monday, 24 June 1985

2:45 - 3:10 p.m. & 3:15 - 3:40 p.m.

#### 1st Sessions

1. Miter Gate Analysis and Design
2. U-Frame Lock Analysis
3. Building Systems Analysis and Design
4. Finite Element Analysis

Monday, 24 June 1985

3:45 - 4:10 p.m. & 4:15 - 4:40 p.m.

#### 2nd Sessions

1. Pile Foundation Analysis
2. U-Frame Basin and Channels Analysis
3. Three-Dimensional Stability Analysis
4. Sliding Stability Analysis

### Training Sessions

Tuesday, 25 June 1985

2:25 - 3:05 p.m. & 3:10 - 3:50 p.m.

#### 1st Sessions

1. Soil-Structure Interaction Effects and Analysis Techniques
2. Retaining and Flood Walls Revised Draft Manual
3. Design Procedure for Determining ANSI A58.1 Wind Load for Buildings

Tuesday, 25 June 1985

3:55 - 4:35 p.m. & 4:40 - 5:20 p.m.

#### 2nd Sessions

1. Simplified Design Equations for Strength Design of Hydraulics Structures

2. Earthquake Analysis and Design of Concrete Gravity Dams,  
ETL 1110-2-303 (in press)
3. Ribbed Mat Slab Foundations Designs

Thursday, 27 June 1985  
3:40 - 4:20 p.m. & 4:25 - 5:05 p.m.

3rd Sessions

1. Review of Structural Engineering Guide Specifications and  
Recent Changes
2. Stability Criteria for the Rehabilitation of Navigation  
Concrete Structures
3. Earthquake Analysis and Design of Intake Towers Draft Manual

ABSTRACTS

## PRESENTATIONS



## KEYNOTE

William N. McCormick  
Chief, Engineering Division  
F&C Directorate, OCE

Mr. McCormick spoke briefly on a variety of topics of interest to the conferees. He discussed the future of the Corps of Engineers in structural design, addressing such things as trends in the use of consultants in design, types of projects anticipated, and future organizational trends. Responding to widespread interest in the status of CE involvement in computer-aided design and drafting, he discussed current status, ongoing activities and future directions in the organization of equipment, and plans for applications. In the category of innovative structural systems, materials, and techniques, Mr. McCormick discussed current thinking in the Congress and how the Army should react to pressures by industry and the Congress to adopt and apply new materials and new concepts. For some time the need for a Structural Engineering Support Center has been voiced. Mr. McCormick is a proponent of this concept and shared his views on its merit as well as its prognosis. Lastly, as a member of both of the Blue Ribbon Panels (the Duschka Panel 1982 and Johnstone Panel 1984), Mr. McCormick reported on the results to-date of those two reviews and the implementation of their recommendations.

## MOUNT ST. HELENS-SPIRIT LAKE OUTLET TUNNEL

Donald R. Chambers  
Structural Engineer  
Portland District

On 18 May 1980, the north slope of Mount St. Helens collapsed following a magnitude 5.0 earthquake. This collapse precipitated almost simultaneously, the most catastrophic volcanic eruption in the continental United States in recorded history. A gravitational landslide ensued, transporting an estimated  $.6 \text{ mi}^3$  ( $2.8 \text{ km}^3$ ) of debris into the upper North Toutle River drainage basin. One of the effects of this "debris avalanche" was blockage of the Spirit Lake drainage outlet by a deposit several hundred feet thick. A lobe of the avalanche rammed through Spirit Lake, raising the lake surface approximately 200 ft. By the summer of 1982, the lake level had risen almost 60 ft higher, increasing the volume of water held back by the debris "dam" from 126,000 to nearly 275,000 acre-ft.

In summer, 1982, a government task force was formed to evaluate the potential hazard. The group determined that the debris dam could not safely pond water above elevation 3,475 because of the composition of the debris avalanche deposit, subsidence potential, and active erosion. It was estimated that Spirit Lake would reach elevation 3,475 by March 1983 if the area received average precipitation and if no preventive measures were taken. The task force concluded that if Spirit lake was allowed to reach that level, the debris dam could be breached. Such an occurrence would cause catastrophic flooding and widespread damage in the Cowlitz Valley. It would also interrupt Columbia River navigation. As an interim solution, a temporary pumping facility was constructed at Spirit Lake. Pumping commenced in November 1982.

Possible alternatives for a permanent solution were considered: (a) a buried conduit; (b) an open channel; (c) a tunnel; and (d) a permanent pumping facility. A number of potential alignments for each alternative were studied.

Because the project lies within the Mount St. Helens National Volcanic Monument, one of the key criterion considered was the degree of site disturbance. Constructibility and cost were other key criteria. Long-term stability was an important concern because some of the alignments crossed the potentially unstable debris avalanche deposit. Downstream impacts, such as erosion

and sediment transport, water quality, and effect on stability of the Spirit Lake and Coldwater Lake Debris dams, were also considered. Finally, the mountain's proximity dictated that each alternative be evaluated for ability to withstand future volcanic or seismic events.

Based on these considerations, the tunnel alternative appeared to be most appropriate. Ultimately, the preferred approach to construction was a straight tunnel extending approximately 8,500 ft from the west side of Spirit Lake to South Coldwater Canyon. A short design period was necessary to allow maximum time for construction during the summer months, so that the tunnel could be completed and the pumping facility removed before the 1985-86 winter season.

The objective of the Spirit Lake permanent outlet project is to lower the lake surface by approximately 20 ft to elevation 3,440, and to maintain this "safe" level with minimal fluctuation. Lowering the present lake surface to elevation 3,440 will require draining approximately 2.8 billion ft<sup>3</sup> (21 billion gal) of water, or about 65,000 acre-ft plus inflow, from the existing 275,000 acre-ft lake.

During preliminary design, various intake and outlet concepts, tunnel cross sections, excavation methods, and lining alternatives were considered. Intake concepts considered included: a conventional lake tap; an open cut and staged intake plug excavation; a shaft and staged excavation (the method used at the Thistle Lake Project in Utah); and an open cut behind a constructed cofferdam. The procedure finally selected was a variation of the shaft and staged excavation concept. The intake structure, a concrete bulkhead containing a single gated opening with a maximum capacity of 500 ft<sup>3</sup>/s, was to be constructed behind a natural rock "cofferdam" left in place in the intake channel.

The tunnel design allowed the use of drill-and-blast or TBM methods. For the drill-and-blast method, a straight-leg horseshoe shape and smooth-wall blasting procedures were required. For the TBM method, the design allowed a range of sizes for a bored tunnel, so that a reasonable number of used TBM's were available to meet the specification (10 ft-10 in. to 14 ft-0 in.). On 7 June 1984, a contract in the amount of \$13,469,247 was awarded to a joint venture of Peter Kiewit and S. J. Groves. The TBM option was selected.

The tunnel is being driven from its downstream end toward Spirit Lake concurrently with excavation for the intake at the upstream end. The TBM

started operations on 28 September 1984. As of 30 January the downstream portal excavation had been completed and 6,400 ft of tunnel had been driven. Intake construction is nearing completion with plans for cofferdam removal in early February 1985. The tunnel is expected to be finished by 1 March 1985. Lake drawdown is scheduled to begin on or before 1 April 1985. It will require approximately 4 months to drain the estimated 2.8 billion ft<sup>3</sup> of water to lower the surface level to elevation 3,440.

For additional information contact: Don Chambers, NPPEN-DB-SA, Comm: 503/221-6906; FTS: 423-6906; or John Sager, NPPEN-FM-G, Comm: 503/221-6460; FTS: 423-6460.

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LOCK AND DAM NO. 26 (REPLACEMENT)  
COFFERDAM TESTING PROGRAM

Richard J. Flauaus, P.E.  
Structural Engineer, LMSD-DA

Reed L. Mosher  
Civil Engineer, WESKA-E

Lock and Dam No. 26(R) is located on the Mississippi River at Alton, Illinois, approximately two miles downstream from the existing Lock and Dam No. 26. The lock and dam structure will be constructed in three stages utilizing steel sheet pile cellular cofferdams founded in sand.

Design criteria for cellular cofferdams are largely based on empiricism and past experience rather than on theoretical understanding supported by field measurements. While these criteria are adequate for the design of most cofferdams, the inherent conservatism necessitated the use of high-strength steel sheet piling within the common wall area for the first-stage cofferdam. Preliminary design of the second-stage cofferdam using first-stage design criteria resulted in all configurations which were overly costly and difficult to construct. These unacceptable designs were due to the severe space limitations and anticipated scour problems associated with the second-stage cofferdam. As a result of these problems, it was decided to attempt to refine the design criteria for cellular structures through the cofferdam testing program.

The approach adopted to develop new design criteria was to design and install a system of instrumentation on two cells of the first-stage cofferdam. The instrumentation selected for the program consisted of 1,600 strain gages at 400 locations, 24 earth pressure cells, 8 inclinometers, 8 pneumatic piezometers, 4 open system piezometers, and 14 target points for trilateration surveys. Readings were made before filling, after filling, after berm placement, during dewatering, during high water, and on a regular monthly basis. As an aid in interpreting and correlating the field data, finite-element models were developed to predict the cofferdam behavior under a variety of loading and material behavior assumptions. After each reading, the field data were analyzed to (a) compute the apparent lateral earth-pressure coefficient, (b) determine the depth of pile fixity and (c) determine the location of maximum interlock tension. Of the 400 strain gage bridges, approximately

80 percent of the gages remained operable and about 65 percent were considered to be producing reliable data after 18 months of service.

As a result of the study conducted on the first-stage cofferdam, significant revisions have been made in the design criteria for the second-stage cofferdam. The classical secant formula for the computation of the common wall interlock forces was modified to predict lower maximum interlock forces, and it was verified that the maximum interlock force does not occur at the dredge line, but at a point approximately 15 ft above the dredge line. The field data supplemented by laboratory tests of sheet piling have greatly improved our ability to construct realistic finite-element models of cellular cofferdams, and have made clear the crucial importance of the flexibility and extension of the sheet pile walls for hoop stress predictions. The changes in the design criteria for the second-stage cofferdam have resulted in savings to the Government of approximately six million dollars.

For additional information on the cofferdam testing program, contact Richard J. Flauaus, LMSED-DA, Comm: 314/263-5524; FTS: 273-5524; Reed L. Mosher, WESKA-E; Comm: 601/634-3956; FTS: 542-3956.

THREE-DIMENSIONAL FINITE-ELEMENT ANALYSIS  
OF A CELLULAR COFFERDAM

Richard J. Flauaus  
St. Louis District

Reed L. Mosher  
Waterways Experiment Station

General. A three-dimensional finite-element analytical model for Lock and Dam No. 26 (Replacement) was developed for the second and third stages of construction. The program was developed at the Waterways Experiment Station with assistance from faculty of Virginia Polytechnic Institute.

Laboratory Testing. The following testing was performed for input into the program:

- a. Load vs Deformation - Uniaxially loaded sheetpiling.
- b. Load vs Deformation - Response of simulated internal cell pressure.
- c. Tests on interlock shear with hoop tension.
- d. Distortion of dye piling connection under load.
- e. Triaxial compression and extension test of cell fill material.
- f. Cubical shear box test of cell fill material.

Program Features. The program will model construction and operation of the cofferdam in stages from initial filling through dewatering.

Comparison with Existing Data. The predicted results are compared with existing field instrumentation data and two-dimensional analytical studies.

For additional information contact: Richard J. Flauaus, LMSED-DA, Comm: 314/263-5524; FTS: 273-5524; or Reed L. Mosher, WESKA-E, Comm: 601/634-3956; FTS: 542-3956.

MUNITIONS STORAGE MAGAZINES, STRUCTURAL FAILURE  
AND EVALUATION OF STEEL ARCH "STANDARD DESIGN"

Kirk M. Price  
Structural Engineer  
Kansas City District

Nearly 300 magazines have been constructed world-wide conforming to the OCE Standard Drawing 33-15-73, entitled "Magazines, Steel Oval Arch, Earth Covered." In the fall of 1984, one of eight igloos at Sunflower Army Ammunition Plant in Kansas collapsed and six of the remaining seven were found to have large deflections at the crown. The Kansas City District was tasked with preparing a technical review of the igloo failure and recommendations for monitoring and evaluating the remaining Sunflower igloos. This report was used as a basis for direction given to affected installations for monitoring and evaluation of other existing igloos.

The Standard Design was developed by an A/E firm using a standard steel-industry culvert shape known as a Long Span, High Profile, Multi-Plate Arch. This shape has a maximum inside width of 28 ft 2 in. and a rise of 14 ft 5 in. The design of the arch followed the somewhat empirical methods traditionally used for design of deep buried culverts which only considered ring compression.

The Kansas City District analyzed the structure and its backfill using a computer program from the Corps Library, CBNTBM. This program was chosen because it can model a curved structure and the supporting backfill can be analyzed as spring supports. The nonlinear spring option of the program allows modelling of initial at-rest pressure on the structure, a low compaction zone of backfill adjacent to the corrugations, and a maximum value of passive resistance.

The following is a summary of the findings of the structural evaluation:

- a. Steel arch buckling is not a problem since compressive stresses are less than 1.5 ksi.
- b. With low backfill stiffness and shallow cover, combined bending and axial stresses can be very high. However, stresses in excess of yield do not necessarily mean the structure will collapse.
- c. If the "as-built" shape of the structure is out of tolerance, the stresses can be greatly magnified.

d. Rotation of lap joints due to compressible mastic waterproofing can also cause stress magnifications.

e. Earth surcharge loads can greatly increase stresses.

f. The horizontal extent of the backfill must be adequate to develop the required passive pressures and required stiffness.

g. Thrust beams or other devices to enhance compaction efforts can significantly lower crown stresses.

The Kansas City District concluded that the major cause of failure was poor horizontal backfill stiffness and that other contributing factors were out of tolerance as-built shape and low lap-joint stiffness.

A procedure for monitoring and evaluating existing igloos was then developed based on the results of the study. A graph and instructions were presented which can be used to determine the required frequency of future monitoring or the need to evacuate the magazine and initiate a repair.

On 8 March 1985, the Chief's Office issued a directive outlining monitoring guidelines for existing structures, stating that the Standard Design had been rescinded, and that no new igloos of this type would be built. At present, various repair options are being investigated for those magazines which have large deflections.

For additional information contact Kirk Price, MRKED-DR, Comm: 816/374-3235; FTS: 758-3235.

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## CTABS80 USAGE IN BUILDING DESIGN

Dennis Bellet  
Sacramento District

TABS80 is a CASE committee computer program for the structural analysis of multistory frame and shear-wall buildings subject to static or dynamic loadings. The Sacramento District has used CTABS80 for the design of five buildings, ranging from small single-story frames to a two-story building with a basement. The special features of each building and why CTABS80 was chosen as the analysis tool will be described, and the benefits and advantages of using this program will be discussed.

For additional information contact Dennis Bellet, SPKED-D-CE, Comm: 916/440-2126; FTS: 448-2126.

## CRATER LAKE HYDROELECTRIC PROJECT

Joseph B. Leeak  
Structural Engineer  
Alaska District

The Alaska District is preparing final plans and specifications for the construction of the Crater Lake Hydroelectric Project at Snettisham, Alaska. The project is located in the Tongass National Forest on the Speel Arm of Port Snettisham, a glacial fiord in southeastern Alaska. The Snettisham Project is 28 miles southeast of Juneau. The Long Lake phase of the Snettisham development was completed in 1973 and included the construction of an intake structure, power tunnel, penstock, underground powerhouse, transmission line, and installation of two turbines and generators. The Long Lake generators have a combined generating capacity of 47 MW. With the completion of the Crater Lake phase, an additional 30 MW will be added to the project's total generating capacity. The Crater Lake phase will require construction of various facilities to include the following:

- a. A 6,020-ft power tunnel from the powerhouse to near the lake bottom through rock consisting of quartz diorite, gneiss, and biotite schist.
- b. A primary rock trap near the lake bottom to permanently store debris when the lake tap piercing blast is completed.
- c. An intake trashrack, installed after the completion of the lake tap blast, to cover the intake orifice.
- d. A wet well-gate structure with a hydraulically operated slide gate and a hoist-operated emergency bulkhead.
- e. A 903-ft steel penstock with ring girder supports at 30-ft intervals.
- f. A vented surge tank.
- g. Installation of turbine, generator, and spherical valve in the powerhouse skeleton bay.

The discussion presents descriptions and considerations necessary for the design of these facilities. It is important that Corps Districts understand the problems and solutions associated with the development of low flow, high head hydroelectric projects in mountainous regions due to the continuing need for additional power-generation facilities.

For additional information contact Joseph B. Leeak, NPAEN-DB-ST, Comm: 907/753-2830.

## GEODESIC DOME STRUCTURE, DESIGN, AND CONSTRUCTION

Peter Lam and Jim Nott  
Structural Engineers  
Alaska District

The Alaska District has recently completed the design and construction of composite facilities consisting of two connected geodesic dome structures at each of four separate radar sites for the Air Force. Locations of these structures are at the remote Air Force stations at Sparrevohn, Cape Romanzof, Tatalina, and Indian Mountain. The dome structures are 24 and 30 ft high on concrete slabs over steel framework for the ground level lower floors. One is for personnel, housing approximately 20 personnel at each of the sites, and the other is for industrial purposes, housing power plants, storage, shops, vehicle storage, and other functions.

The structural configuration of the domes minimizes the exterior surface area exposed to the extreme arctic climate. The domes are designed for a 148 lb per sq ft ground snow load, basic wind speed of 120 mph, seismic zone 3, and temperatures ranging from  $-70^{\circ}\text{F}$  to  $+100^{\circ}\text{F}$ . At two sites, the only access is by aircraft. The industrial dome is 100 ft in diameter with the residential dome 90 ft in diameter.

At Indian Mountain, the building foundation is permafrost. Thermoprobes and insulation are installed under the buildings to shield against the heat from the buildings and maintain the permafrost, avoiding foundation displacements.

The domes are fabricated from extruded 6061-T6 aluminum, 6 in. deep, wide flange struts, approximately 8 ft long, forming triangular structural framework, covered by composite sandwich panels, with a 16-gage 3003-H16 aluminum outer and inner surfaces and 5 in. of insulation between the aluminum sheets.

The total cost of construction at the four sites was \$42 million and took two years, which included time for shop work, shipping, and site erection. Material and erection costs of the 8 aluminum domes were approximately \$3.3 million. The field work took approximately 10 months. The dome parts of the structures were erected by a five-man team, working 8 weeks, 7 days a week, 12 hours a day.

For additional information contact Peter Lam or Jim Nott, NPAEN-DB-ST,  
Comm: 907/753-2828; FTS: 907/753-2828.

STATUS REPORT COMPUTER-AIDED STRUCTURAL  
ENGINEERING (CASE) PROJECT

Dr. N. Radhakrishnan  
Chief, Automation Technology Center, WES

In the recent years, substantial progress has been made in developing user-oriented programs for the structural engineers in the Corps. Most of this development has been done under the Computer-Aided Structural Engineering (CASE) Project. CASE work is done through various technical task groups set up one for each particular structure. The task groups consist of design engineers from the field offices. Both Civil Works and the Military projects are included in the project. Another unique feature of CASE is consideration of geotechnical aspects by involvement of geotechnical as well as structural engineers from the various Corps FOA's.

The CASE Project has produced 34 programs and 34 reports. Five ETLs have also been published. Guidelines on Computer-Aided Drafting for Structural Engineers and the use of finite-element methods in design/analysis of Corps-type structures are also being developed. Final programs are available for the design/analysis of building systems, T-walls, sheet-pile walls and cells, culverts, conduits, bridges, gravity dams, pile foundations, miter gates, bearing capacity and settlement of footings, stability analysis of general 3-D structures and others. Programs are under development for the design/analysis of U-frame locks, channels and basins, 3-D lock walls, intake structures, etc. These programs are being used extensively by the field offices. For the past five years the CASE programs have been used over 70,000 times with all Corps offices using at least one CASE program. Even further, a survey indicates that the CASE programs were used on over 300 Corps projects by 50 Corps offices. Thirty-five short courses on the use of the developed programs have been offered.

The project emphasis on involving ultimate users of programs in the initial criteria development and pre-release evaluation has borne fruits and the concepts will be outlined. The CASE project products will be an aid for preparation of future structural designs. By optimally matching the talents and experience of design engineers in the field offices and Resource and

Development (R&D) resources, the project has demonstrated that costly duplication of efforts can be prevented and healthy technology transfer can take place between the OCE, field offices, and the R&D installations.

For additional information contact Dr. N. Radhakrishnan, WESKV, Comm: 601/634-2527; FTS: 542-2527.

## COMPUTER-AIDED DESIGN AND DRAFTING STUDIES

John E. Naeger  
Civil Engineering Technician  
Functional Manager CADD System  
St. Louis District

The purpose of this talk is to provide information on the experience of the St. Louis District in implementing its computer-aided design and drafting system. We have defined such a system as all hardware and software required for a computer-oriented engineering drafting and editing system.

CADD has a bright future in the Corps, especially in the Structural area, with the introduction of specialized software packages such as Reinforced Concrete Design and Finite-Element Analysis. The advent of stand alone engineering workstations will make these programs easily accessible to all engineers.

We feel that our implementation experience is probably typical of what others can anticipate. The St. Louis District has demonstrated that CADD is an efficient and cost effective means of accomplishing its design and drafting function.

For additional information contact John Naeger, Comm: 314/263-5856;  
FTS: 273-5856.

DEMONSTRATION OF COMPUTER-AIDED DESIGN AND  
DRAFTING SYSTEM CAPABILITIES

Robert Holt and Roger Hoell  
St. Louis District

This demonstration shows the capabilities and advantages of a CADD system for developing design drawings ranging in scope from a letter report to final plans. It includes creating basic-line geometries, building and use of standard cell libraries, manipulation of views, and data management by use of level overlays. This presentation also addresses checking of structural interferences (rebar with embedded items, etc.) and fits of electrical and mechanical items within the structural geometry.

The interaction of the design engineer with the creation of working and final drawings is covered both from the standpoint of present capabilities (in the St. Louis District) and future intentions. Potential interaction between the in-house CADD system and main-frame time-sharing computers, in-house micros, and in-house engineering work stations is also addressed.

For additional information contact Robert Holt or Roger Hoell, LMS-DA,  
Comm: 314/263-5693; FTS: 273-5693.

## MICROCOMPUTERS IN STRUCTURAL ENGINEERING

William A. Price, III  
Chief, Engineering Applications Group  
Automation Technology Center, USAEWES

This paper is an anthology of user comments and other information on the role of microcomputers in structural engineering. It has three main segments.

The first segment attempts to set microcomputers used for engineering applications into proper perspective, discussing some common misconceptions, defining a microcomputer, and restating the truism that even though you can do something, it doesn't necessarily mean that you should do it.

The second segment reports the results of a 1985 Corps-wide survey on the use of microcomputers for engineering applications in the Corps of Engineers.

The final segment is a report on selected successes in Corps field offices and some comments on what you might consider when deciding what is best for your office.

The speaker can be reached at Comm: 601/634-3645; FTS: 542-3645 for comments or questions. Correspondence should be to the author, ATTN: WESKA-E.

THE ENGINEER'S RESPONSIBILITY FOR  
STRUCTURAL INTEGRITY AND SAFETY

General Robert D. Bay (USA Ret)\*  
Black and Veatch E/A

It seems like only yesterday when my now grown sons were very small. I have a vivid memory of a catastrophe that took place while I was babysitting. My wife went shopping, leaving a cake she had baked for company the next day on the kitchen counter so the icing could set. The last thing she said was for me to keep an eye on everything, especially the boys. We had great fun playing in their room for about an hour while she was gone. Then there was this awful scream, and we all ran to the kitchen. There was my wife, the dog with icing from head to tail, and cake everywhere. I remember those piercing words, "I trusted you to see that this didn't happen." I had concentrated on keeping the boys and thought I was discharging my responsibility. I forgot there were other areas she entrusted to my care.

As structural engineers, the public places their confidence in us to provide them with useful, safe structures. In recent months, there have been apartment house failures during construction in England, a hotel in Kansas City had a walkway collapse, a deck slab dropped from a bridge in New England, a dam failed out west, an auditorium roof collapsed, and, more recently, a roof gave way in a swimming pool in Switzerland. Almost every week I read accounts in the Engineering News Record concerning a project with a structural problem. We are all beginning to hear piercing words, questions that ask about our responsibility in the structural integrity of projects.

Each year, the American Society of Civil Engineers (ASCE) selects a major theme for emphasis. The theme this year and next is "Quality in the Constructed Project." As President-Elect, I travel quite a bit and have gathered lots of opinions and observations on this theme from those closely associated with construction.

The first group I talked to were Owners...and let me remind you that one of the biggest owners in the United States is the government itself. What did these people have to say? They said, "We expected quality construction and

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\* Mr. Bay was ill and unable to attend the conference but provided a copy of his presentation for this report.

the final project did not meet our expectations." They blame designers, contractors, construction managers, and field inspection for all the project flaws. They complained that even the slightest change in the project became a big ticket change order. They claimed the delivered facility had high early maintenance costs. Finally, they said we are in the middle in litigation and insurance claims.

I next spoke to the designers. In this category I included architects, civil, electrical, and mechanical engineers, and others whose profession is design. However, given the nature of this audience, I will confine most of my remaining remarks to the structural engineer. What did the designers say? They said: Cost is always a factor in contracting for services. The money available limits the examination of all the possible alternatives. During the negotiation phase, owners are not sure of requirements and after the contract is signed they want to make changes in the scope and level of performance of the structure without any additional compensation. Time is money and deadlines promote fast track programs. This causes internal reviews to suffer due to the lack of time and jobsite change orders result. Owners and contractors gang up on designers during construction to make radical changes in the project to save dollars--subtle little changes in materials, concrete cure time, backfield compaction requirements, etc. These all sacrifice the integrity of the structure. The designers claim they're never consulted on these changes. They don't have a role at the jobsite. Owners don't want to pay for the designers' involvement at the jobsite. They felt they were caught in the middle.

When I talked to construction managers, they indicated that their job was to keep the project on schedule and to see that it is built in accordance with the plans and specifications. They said the owners didn't really want to pay for quality construction management and habitually beat them down on their contract price. They feel they can't adequately man the jobsite and when a problem occurs on the project, everybody clams up on advice of attorney for fear that they will commit themselves to some sort of solution that might expose them to some liability. Even when there is a ready solution to a problem, it doesn't come forth very quickly. Finally, the construction managers said, "We are caught in the middle."

The next group I talked to were contractors. They were quick to remind me that they were just as interested in quality as anyone and that their

reputation and future work depended upon satisfied clients, and I admit that I agreed. They said the drawings for the project were not clear. They're not even correct from sheet-to-sheet requiring job-site changes and delays that resulted in the need for change orders. They indicated that plans call for complicated construction techniques. When they requested changes to simplify the construction, they were always challenged by the designer and the owners. They felt sometimes, even though it was a good idea, the designers weren't willing to admit there was a better way and frequently covered up their oversight by indicating that they would permit the change provided there was a credit, even when it wasn't warranted. The shop drawings that were required as part of the contract were always simply stamped received, giving no indication that the materials scheduled on that shop drawing were satisfactory. The contractors stated, "We're not the experts. We feel like we're between the suppliers on one side and the owners and designers on the other, and someone needs to make a decision as to whether the materials we intend to use are fully satisfying the job requirements." They said, "We never see the actual designer at the jobsite."

Now, I would like for you to note, all of the participants on the project pointed the finger to all the other players on the project team and everyone felt that they were somewhere in the middle. It soon became apparent to me that each player has much to do to improve jobsite quality. But, I'm here today to focus on our responsibility as structural engineers. What practices do we follow that need to be improved to keep others on the team from pointing to us? Let me indicate at the beginning here maybe, just maybe, we are so conscious of our liability exposure that we listen to lawyers too much and maybe we might be guilty of practicing crutch engineering. Crutch engineering is the practice of discharging responsibility by making others responsible for those areas where decisions could cause liability exposure. In other words, any practice to evade liability. Let me give you eight examples of crutch engineering that structural engineers may be guilty of.

The specs and drawings that are prepared call for rebars to be in accordance with the ACI building code. Let me ask, what does a draftsman for a reinforcing bar fabrication shop, who took the job on price, know about the special conditions in walls, beams, and slabs where the rebar is needed? There are critical points in all construction that only we design engineers

know about since we designed the job, and we must provide guidance if we are to assign final product design to others.

We call for all connections of structural steel to be in accordance with standard American Institute of Steel Construction connection details. What does a structural fabricator detailer know about the loads coming into a column from floor beams or the need for wind bracing? Are we guilty of not providing enough guidance to properly design that connection?

On nearly every project where a major failure has occurred and mentioned by me earlier, there was a joint or connection problem. As I see the problem, it's not only the connection but the interface between the new construction materials available, computer software programs for design applications and new design systems being developed that need attention. I call that the crease. In football, the quarterback on the offensive team tries to hit the pass receiver somewhere in the crease, because that's where the defensive zones come together and it's the place where the defensive backfield may not be coordinated for adequate coverage. In the design profession, the crease is the place where materials, software programs, and design systems come together that are not normally included in any design process and need attention or there may be problems.

There's too much catalog design. Often we see on the drawings statements saying, "In accordance with the manufacturer's recommendations." What do the manufacturers know about the other materials with which their product interfaces?

Shop drawings. Much has been said on the subject, but it suffices to say you can't stamp receive and escape liability. So it's best to exercise your professional responsibility in a proper way and make a complete review of those drawings.

Constructability reviews of our designs at the 20 or 30 percent stage is essential but not often completed. Why not have the experience of a builder interface with designers who are good at design, but may not have all the construction site experience that those people have? I'm confident that practical changes would avoid complicated required construction techniques and the high project costs that accompany such requirements.

A complete quality control review of all calculations, drawings, and specifications by an independent group is necessary but, too often, not completed or required by in-house designers.

We do not plan and insist on site involvement as design engineers.

What we need is a better understanding of the role and scope of services normally provided, a sort of standard of care for each of the players on the construction project team. I'm pleased to report that ASCE has formed a committee and is commencing to produce a manual of professional practice for professional services normally provided by the civil engineer. Adoption is planned on a consensus basis following broad industry review by all the players. When adopted, it could become a standard for such services and be the first of a series of publications outlining the scope of services for each of the players--architects, mechanical engineers, electrical engineers, contractors, construction managers, and field inspectors.

But the greatest contribution we can make right away as structural engineers is to stop practicing crutch engineering and to be a part of a team that communicates with one another all the time; to understand that quality is a primary responsibility of all connected with the project at all times; cease to rely on phrases, disclaimers, contract documents, plans, and specifications by taking design responsibility for all our work and doing whatever design is necessary; and finally, when there is a problem on the job, come together as a team and solve it by not holding back on a recommendation or a solution for fear of personal liability. Add integrity to all that, and I guarantee you continued client and public confidence in entrusting to our profession the adequacy of the final constructed project. Thank you.

OLD RIVER CONTROL AUXILIARY STRUCTURE POSTTENSIONING  
SYSTEM (FIELD EXPERIENCE WITH DYWIDAG GRADE 160  
THREADBARS ON TRUNNION ANCHORAGES)

Thomas Hassenboehler, P.E.  
Supervisory Structural Engineer  
New Orleans District

Alan Schulz, P.E.  
Structural Engineer  
New Orleans District

The Old River Control Auxiliary Structure is located on the Mississippi River approximately 50 miles Northwest of Baton Rouge, Louisiana. The structure has been under construction since November 1982 and concrete placement is 85 percent complete as of January 1985. The auxiliary structure will be used in concert with the existing Old River Control structures to maintain a 70-30 flow distribution between the Mississippi River, Red River, and Atchafalaya River systems.

The Auxiliary Structure is a tainter-gated dam structure composed of 7 piers and 6 gatebays each with a 62-ft clear opening. The gates pivot on hinges supported by concrete trunnion girders that are anchored by a post-tensioning system into the piers. Very large posttensioning forces are required to resist the downstream horizontal thrust of the river during extreme conditions. The posttensioning system used was a DYWIDAG threaded bar anchorage type with Grade 160 properties (160 ksi ultimate tensile strength) conforming to American Standards for Testing Materials A722. This is, to our knowledge, the first time bars of this strength have been used on a Corps tainter gate anchorage. Eighty-four bars, each 1-3/8 in. nominal diameter, were used per half pier to anchor the trunnion girder into the pier. The trunnion girder was posttensioned along its own axis using 78 bars. Stresses at initial seating (lockoff) were 70 percent of the ultimate bar strength. Plate-type anchorages were used on the trunnion girders and on the exposed ends of the pier tendons. Bell anchorages were used on the embedded (cutoff) end of the pier anchorages.

During installation of the bars, they are isolated inside galvanized ducts to form a void for posttensioning. The bars are stressed by hydraulic jacks and, after verification testing, are grouted into the piers primarily

for corrosion protection. The grout used was a portland cement base with an expansive agent (aluminum powder) to compensate for shrinkage.

Subsequent to stressing and verification testing of the initial pier on the auxiliary structure and shortly after grouting, four posttensioning bars broke in a brittle type failure mode. An extensive testing and evaluation program was undertaken to determine the cause of failure. The chemical and mechanical properties of the bars were checked as well as all constituent materials of the grout. Results of these tests and comments on suggested updates on Corps guidance will be presented. The probable cause of failure was found to be "hydrogen embrittlement" which can result from the combination of the normal high tensile stress levels in the bars and the particular bar metallurgy that was present in the problem bars. This bar metallurgy is due to the relatively high "hardness" values (due to improper chemistry which resulted in very high tensile strength/lower ductility) along with electrolytically generated hydrogen gas from the grouting material.

Various remedies were incorporated to correct the problem bars. This included replacement of some bars and revisions to the grout mixture, which involved reduction of the aluminum powder fraction and substitution of different type grouts without aluminum powder. Details of results of these remedies will be discussed.

For additional information contact Tom Hassenboehler or Alan Schulz, LMNED-DD, Comm: 504/838-2660 or 2652.

PROTOTYPE DYNAMIC TESTING OF THE RICHARD B. RUSSELL  
CONCRETE GRAVITY DAM

Vincent P. Chiarito  
Research Structural Engineer  
Waterways Experiment Station

Forced vibration tests of the Richard B. Russell Dam were conducted before and after the reservoir was impounded. This was a rare opportunity to experimentally measure the dynamic properties of a concrete gravity dam and determine the effects of the hydrodynamic interaction. The dynamic properties determined include the following: the natural frequencies, mode shapes, modal damping ratios, relative joint motions, and hydrodynamic pressure distributions. The first low-level forced vibration test was conducted during January and February 1982, and the second test after the reservoir was impounded during June and July 1984. The experimentally measured dynamic properties were compared to two-dimensional (2-D) and three-dimensional (3-D) dynamic finite-element (FE) modelling assuming no reservoir was impounded. Presently, 3-D dynamic FE analyses are being conducted to model the added mass effects of the reservoir.

For additional information contact Mr. Vincent P. Chiarito, WESSS-R,  
Comm: 601/634-2714; FTS: 542-2714.

### THREE-DIMENSIONAL ANALYSIS OF ENGLEBRIGHT ARCH DAM

John W. White  
Chief, Civil Design Section A  
Sacramento District

The US Army Engineer District, Sacramento, is conducting a seismic evaluation of Englebright Dam under the provisions of the Dam Safety Assurance Program (SAP). This paper provides an overview of the finite-element analysis of the dam, with emphasis on computer modeling and recommendations of computer methods.

Englebright Dam is a constant-angle, concrete circular arch, overflow dam constructed in the early 1940's on the Yuba River in California. It is 260 ft high with a crest length and width of 1,142 and 21 ft, respectively. A central overflow section in the dam is depressed 15 ft with 5 bays for a total net length of 486 ft. A slight overhang on the crest of the dam permits overpassing water to fall clear of the base of the dam. Aeration piers are provided to ventilate the nappe.

Englebright is located in Seismic Zone 3, an area of major seismic hazard. The dam is, thus, potentially subject to high seismic loads as well as significant probable maximum flood overtopping of the entire crest. Based on fault studies of the region, the maximum credible earthquake is a 6.5 magnitude at 8 km, yielding 0.5 g acceleration at the dam site. The horizontal components for two representative accelerograms for this site were developed by Drs. Bruce Bolt and Bolton Seed of the University of California at Berkeley. A vertical component was determined from these accelerograms.

Dynamic tensile splitting tests were made on core samples to determine allowable tensile stresses for seismic loads. Dynamic stress increases were not as expected, based on similar work performed by the State of California. Additional coring and testing are required to address questions regarding procedures and aggregate size in the dynamic tensile splitting tests.

GTSTRUDL, developed by the GTICES Systems Laboratory of the School of Civil Engineering at the Georgia Institute of Technology, was initially used for static and dynamic analyses. Program constraints limited the inclusion of the foundation in the model, and forced modeling of hydrodynamic interaction by the "virtual mass" approach developed by Westergaard. Results from these

analyses indicated significant overstressing. At that point, several difficulties were encountered with the use of GTSTRU DL. Stress time histories cannot be produced with the current version of the program. GTSTRU DL is also unable to accurately depict the dam-foundation interaction at the interface. The size of one preliminary, 20-node computer model severely taxed the program's file handling, one feature which is an asset in this type of analysis.

Sacramento District (SD) therefore is using a preliminary version of Earthquake Analysis of Concrete Dams (EACD). This program was developed by Dr. Anil Chopra of the University of California at Berkeley for use in the finite-element analysis of a concrete dam, gravity or arch. EACD uses Fast Fourier Transform and substructure techniques to accurately model reservoir-dam-foundation interactions at each interface. The program was also used to check the data obtained from static analyses by GTSTRU DL. EACD will be used to perform the final dynamic analyses of Englebright Dam.

The static loads consist of head pressure, tail water pressure, silt pressure, and temperature loading. For static loads, the silt is treated as a fluid slightly more dense than water. Dynamically, the silt must be treated as water because EACD does not handle sediment loading. Temperature loading conditions were developed, and supplemental laboratory testing begun to determine the appropriate value which takes into account concrete diffusivity. Parametric studies are planned to analyze the resultant stress levels with variation of the diffusivity of the concrete.

In addition to the static loading under the normal reservoir pool elevation, a surcharge loading condition will be analyzed to determine the dam's performance during the probable maximum flood (PMF). Under the PMF the entire crest would be overtopped by 16 ft. The analysis will also include additional negative pressure loading distributed on the downstream face of the dam to equal the vapor pressure of the water because it is assumed that the nappe will not be aerated. Vibration loading from the PMF will not be considered in this analysis.

The dynamic analyses using EACD were initiated at the time of this writing. The element library for the program includes both thick-shell and variable-node solid or brick elements. Studies are ongoing to determine the number and type of elements to be used in the final model. Convergence studies are being conducted with the use of "base models." The "base model" is the dam with a rigid foundation and no reservoir loading. Such models are being

developed for 20-node bricks, thick-shell elements, and a double layer of bricks in the lower, thicker portion of the dam. The changes in mode shapes and frequencies will be evaluated for convergence and an appropriate dam model determined for use in the final dynamic analysis.

Base models with 5, 7, 9, and 11 modes will be used for time-history computer runs on EACD. The resultant stresses will be used to determine the appropriate number of modes to be used in the full dam-reservoir-foundation runs. The foundation portion of the model will equal one dam height.

Input parameters for EACD consist of the number of modes of dam vibration, the constant hysteretic damping factor for the dam, the wave reflection coefficient of the reservoir bottom and sides, Young's modulus of elasticity for the foundation and the dam, and Poisson's ratio for the concrete and the foundation rock. As the GTSTRU DL model, a small nonzero density value is used for foundation elements to prevent the occurrence of zeros on the diagonal of the mass matrix.

The wave reflection coefficient is equal to the amplitude of the reflected hydrodynamic pressure wave divided by the amplitude of a vertically propagating hydrodynamic pressure wave incident on the reservoir bottom. The wave absorption of the reservoir bottom significantly affects results for wave reflection coefficients between 1.0 and 0.9. Changes are coefficient value below 0.9 which would be justified for a slight to moderate siltation in the reservoir. Initially, a coefficient value of 0.9 will be used in the dynamic analyses. Lower values may be considered, based on the results of these analyses.

Parametric studies on the full dam-reservoir model will be limited to concrete diffusivity, the modulus of elasticity of the concrete, and the modulus of elasticity of the foundation. Only two parametric studies are anticipated for the rock modulus: the mean value, and a softer, lower bound determined by Corps geologists. Decisions regarding parametric studies for the concrete modulus are pending completion of SD's supplemental concrete testing program.

Conclusions from computer models to date are limited, but preliminary studies have yielded much information regarding the software requirements to adequately perform a finite-element analysis of a concrete arch dam. GTSTRU DL possesses excellent graphics and file handling capabilities, both necessary for this type of study. However, the program is limited. Stress time

histories cannot be produced with the current version of GTSTRU DL. The program is also unable to accurately depict the dam-foundation interaction at the interface. The size of a model can overtax the program's file handling. EACD, on the other hand, offers Fast Fourier Transform and substructure techniques to accurately model reservoir-dam-foundation interactions at interfaces. Therefore, EACD is able to refine the estimated values of high stress levels encountered in the preliminary analyses by GTSTRU DL. The use of GTSTRU DL for static analyses of seismic loads, and of EACD for dynamic analyses based on data obtained from the GTSTRU DL analysis appears at this point in time to be the best method of performing a finite-element analysis of a concrete arch dam such as Englebright.

For additional information contact John White, FTS: 460-2070.

## TACTICAL EQUIPMENT SHOPS, DESIGN AND LAYOUT STUDY

Gary R. Close, P.E.  
Structural Engineer  
Savannah District

Savannah District recently conducted a study for OCE to develop design criteria and recommend a layout for Scheduled Maintenance Facilities (SMF's) to be included in future Tactical Equipment Maintenance Facilities. The purpose of the SMF's is to provide an area to perform scheduled maintenance on tactical vehicles that will get the maintenance activities out of the elements and place all unit maintenance under one roof. This paper will present the results of that study and show how the criteria was incorporated on an in-house design project.

The district formed a study team assisted by representatives from the US Army Construction Engineering Research Laboratory and the US Army Ordnance School, Aberdeen Proving Ground, Md. The team interviewed maintenance supervisors at Fort Lewis, Wash., and Fort Hood, Tex., and observed scheduled maintenance activities for a variety of units and vehicles.

After considering as many physical and operational constraints as practical, the team formulated four different schemes to be considered for the study. One scheme uses the bay layout for the current Combat Electronic Warfare Intelligence (CEWI) shop design with added features to enhance the scheduled maintenance activities. Another scheme presents a somewhat austere design that provides only minimum protection from the elements, but it also has features to enhance the maintenance activities. A third scheme is similar to the recommended scheme except it incorporates a single interior column in the center of the frame.

The recommended scheme is a three-bay SMF module with bay dimensions of 32 x 64 ft (CEWI) but with no interior columns. The module contains an inspection/maintenance pit in one bay. The SMF module is provided with the following features:

- a. Lighting and electrical supply.
- b. 7-1/2 ton overhead crane.
- c. Centralized high pressure, hot water washing system with quick disconnects in convenient locations.
- d. An exterior grit/oil/water separator.

- e. A waste oil storage tank.
- f. Unit heating system.
- g. A centralized lubricant storage and dispensing system.
- h. A storage area for scheduled maintenance items (filters, etc.)
- i. A plumbing system including water and compressed air.
- j. An inspection/maintenance pit with associated lighting, ventilation, electrical, waste oil collection systems, and bulk dispensing outlets.
- k. An optional power-pack dolly to provide greater flexibility of SMF operations.
- l. A waste antifreeze collection system.

The results of this study will impact structural engineers in all districts doing military design and review. Presentation of this paper will help clarify the design criteria for scheduled maintenance facilities.

For additional information contact Gary R. Close, SASSEN-DS, Comm: 912/944-5587; FTS: 248-5587.

INVESTIGATION OF STRUCTURAL DAMAGE OF  
TACTICAL EQUIPMENT SHOPS,  
FORT STEWART, GEORGIA

W. T. Cheung  
Structural Engineer  
Savannah District

The purpose of this talk is to present the history and outcome of the investigation into the structural damage of rigid frames in a Tactical Equipment Shop at Fort Stewart in 1982. Evidence presented and conclusions drawn from this investigation emphasize the often overlooked role that improper fabrication and/or erection of structural steel plays in structural failures. It also stresses the necessity of closely monitoring contractor-construction procedures. As will be seen, good construction ultimately comes not from Contractor Quality Control (CQC) but from a comprehensive Quality Assurance Program.

The Contractor for the Tactical Equipment Shops at Fort Stewart, Ga., notified the Area Engineer on 7 April 1982 that winds had damaged the structural frames of one of the shops. The damage consisted of bent flanges in three of the rigid frames in Shop No. 3, one of the four buildings in this project.

Initial investigation by the Savannah District Structural Section, and an independent investigation by our architect-engineer consultant revealed that the damage was not indicative of wind loading, but was of the type caused by improper handling and erection of the steel. The Contractor was directed to correct the damage and proceed with construction. The Contractor then hired a consultant to analyze the structure. In turn, the consultant retained detailed structural analysis of the building.

At the request of the Contractor a meeting was held on 19 May 1982, at which time his consultants presented the initial results of their investigation. Their analysis was based on a 3-D space-frame analysis of the building and a finite-element analysis of our typical building frame. They concluded that frames were inadequate to resist the design wind speed of 90 mi/h, since the finite-element analysis indicated stresses of 90 ksi. They further stated that the structure was inadequate and unsafe. The meeting ended with the

understanding that the Contractor would stop work on building No. 3 until a detailed review of their work could be made by the District.

The district completed a 3-D frame analysis, finite-element analysis (FEM), and a comprehensive review of the Contractor's report. The maximum frame stress resulting from the District's FEM was 27 ksi, not the 90 ksi as claimed by the Contractor. Consequently, the analysis showed that the building was adequate for the design wind speed of 90 mi/h. Wind records showed that the highest wind since erection of the buildings was approximately 40 mi/h which further supported the District's position that the damage was not due to wind. To further substantiate our position, the Fritz laboratory at LeHigh University was retained to complete an independent investigation. Their investigation consisted of a site visit to the project and laboratory testing of representative beams loaded to failure. Their conclusion was that the damage at Fort Stewart was not the type expected as a result of wind loading, but rather the type caused by mechanical damage. The results of the District's investigation was presented to the Contractor and his consultant at a meeting in the District Office on 7 June 1982. At that time the Contractor's consultant submitted a revised FEM analysis showing maximum stresses reduced from the initially reported 90 ksi to approximately 27 ksi which agreed with the District's analysis. The Contractor was again informed that the design was adequate to withstand the design wind loads and that the damage was due to erection and/or fabrication errors. The Contractor was directed to submit his proposed method for repairs.

On 23 July 1982, another meeting with the Contractor was held in the District Office to further discuss his claim of an inadequate design and to issue firm direction regarding his proposed method of repairs to the building. The Contractor did not support his previous claims regarding the design and agreed to drop all existing and future claims pertaining to the damage. An agreement was reached on the extent of repairs required.

For additional information contact Hal Thomas, SASSEN-DS, Comm: 912/944-5571; FTS: 248-5571; or Martin Bandy, SASSEN-DS, Comm: 912/944-5567; FTS: 248-5567.

SIMPLIFIED LOAD FACTORS FOR ETL 1110-2-265,  
STRENGTH DESIGN OF HYDRAULIC STRUCTURES

Clifton C. Hamby, III  
Structures & Civil Section  
Vicksburg District

The equations presented in ETL 1110-2-265 are written in a form which can be used for analysis of existing structures; however, the same equations cannot easily be used for design. There are, at present, no published methods for design of concrete members in accordance with ETL 265 subject to both flexure and significant axial load. The Vicksburg District has developed design equations using principles of interaction diagrams which allow for the direct solution of the area of steel by hand computation. These equations can be used for the design of flexural members, flexure plus axial compression and flexure plus tension. This presentation would show the theoretical development of the method and a summary of the steps used to design members in accordance with ETL 265. Because of the technical nature of this presentation it might be more appropriate as a special topic for interested participants instead of a presentation for the general assembly.

For further information contact Mr. Clifton C. Hamby, III, LMKED-DS,  
Comm: 601/634-5536; FTS: 542-5556.

## MILITARY PROJECT DESIGN

Allan G. Wesley & Larry Cozine  
Structural Engineer & Architect  
Louisville District

In 1982 the Louisville District was assigned a military mission covering Army and Air Force facilities in Kentucky, Illinois, Indiana, Ohio, and Michigan. Diversified requirements in size, function, and appearance of Military Structures have resulted in the design of a variety of structural systems.

UEPH, CHANUTE AFB, Ill., issued in August 1983, is a partial site adaptation that employs masonry-bearing walls that act as shear walls to take lateral loads. Some upgrading of the site-adapted design was necessary to meet current seismic criteria.

Flight Simulator Building, Fort Campbell, Ky., issued in December 1983, employs two structural systems. A pre-engineered rigid-frame metal structure forms an exterior shell that houses the simulator area and a structurally independent two-story classroom/office structure composed of cold-formed steel framing and concrete masonry.

AVUM Hangars, Fort Campbell, Ky., scheduled for issue in April 1986, are braced steel-frame structures that house hangar bays for the UH-60 Blackhawk Helicopter and associated shop and office spaces. Steel trusses span the hangar bays. Concrete masonry partition walls form the shops and offices. A concrete masonry firewall separates Hangar Bays from shop spaces.

Guest Housing Facility, Ft. Benjamin Harrison, Ind., issued September 1984, provides 16 Lodging Suites that utilize a standard modular design. Cold-formed steel framing is used for partitions and exterior load-bearing walls. Concrete masonry load-bearing walls provide fire separation and shear resistance.

Battalion Headquarters and Classroom Building, Ft. Campbell, Ky., issued October 1983, represents an ARHOC prototype designed to incorporate future aesthetic continuity for future growth. Load-bearing masonry walls support steel roof beams and open web joists.

For additional information contact Allan Wesley or Larry Cozine,  
ORLED-D, Comm: 502/582-6516/6279; FTS: 352-6516/6279.

## POST-AND-PANEL-TYPE RETAINING WALL

Bryon K. McClellan  
Chief, Structural Section - ORL

The Mill Creek Channel Improvement Project is located in southwestern Cincinnati, Hamilton County, southwestern Ohio. Section 2 is located approximately 3 miles upstream from the mouth of the Mill Creek beginning at Sta. 1145+00 and ending at Sta. 1263+00 (11,800 linear feet). The Post-and-Panel-Type Wall (P&P Wall) comprises 4500+ bank-ft of the project.

The portions of the stream covered here required some type of vertical retaining wall because of the proximity of mainline railroad tracks on the east (left) bank and of various structures and streets on the west (right) bank. The original Design Memorandum for the project called for a steel sheet pile retaining wall, cantilevered wherever possible, and anchored with wales wherever necessary. However, subsequent subsurface exploration showed that a stiff clay till or top-of-rock elevation was higher than the stream invert, which precluded the driving of sheet piles. After several alternatives were considered, the P&P Wall was chosen.

The P&P Wall consists of H-piles encased in 2-ft diameter caissons, space 6 ft on centers, with pre-cast concrete panels spanning between the piles. The caissons are filled to within a foot of the channel invert, with the pre-cast panels starting at the top of the concrete. Because of the height of the wall, rock anchors were drilled beside the H-piles to add resistance to the horizontal earth pressure.

The contract was awarded for construction of this project in May 1984 with construction beginning soon thereafter. Construction is scheduled to be completed in the spring of 1986. This type of wall construction was readily accomplished under restrictive conditions, and is recommended for similar applications.

For additional information contact Bryon K. McClellan, ORLED-D, Comm: 502/582-5784; FTS: 352-5784.

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STRUCTURAL AND ARCHITECTURAL DESIGN  
FEATURES OF FLOOD WALLS

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Telephone 504/838-2645

Levees and flood walls are used in southern Louisiana to protect cities from river floods as well as from hurricane induced tidal floods. The flood protection is presently being constructed under, among others, the Mississippi River and Tributaries Flood Protection project; the Lake Pontchartrain, Louisiana and Vicinity Hurricane Protection project; and the Atchafalaya Basin Flood Protection project.

This presentation provides information on the experience of the New Orleans District with the design features and architectural finishes associated with flood wall design in urban areas. The types of flood walls used are cantilever "I-walls" and pile supported, inverted "T-walls." The selection of "I" versus "T" walls is made by considering wall deflection, settlement, and stability. Structural steel floodgates are used to cross highways, city streets, and railroad tracks with special types of pile supported "T-walls" used as gate monoliths. The types of floodgates used are "Swing Gates," "Bottom Roller Gates," "Overhead Roller Gates," and "Miter Swing Gates." The selection of the type of gate required is made by considering space limitations, width and height of opening, ease of operation, and aesthetics.

For additional information contact Jorge A. Romero, LMNED-DD, Comm: 504/838-2645; FTS: 504/838-2645.

## ROLLER-COMPACTED CONCRETE FOR ELK CREEK DAM

Dennis R. Hopman  
Chief, Concrete Control Section  
Portland District

Since construction of Willow Creek Dam in 1982, Elk Creek Dam in southwest Oregon is the only Corps of Engineers dam to be constructed using roller-compacted concrete (RCC) that has passed beyond the planning stage to engineering and design. Design and construction concepts for Elk Creek Dam are considered to constitute the current Corps of Engineers criteria for RCC for dams. Because of the size and purpose of the structure, design concepts were established to assure a low rate of seepage through the dam, particularly at the lift joints, to provide a smooth and durable exposed surface for the full height of the upstream face and spillway, and to increase the rate of placement of the RCC.

Procedures to improve watertightness, each to act independently and with each other to prevent seepage, include construction of a three-ft-wide conventional concrete upstream face, extensive lift-joint treatment to eliminate voids and improve bond, and rapid placement of RCC at low temperatures to reduce the amount of cracking due to thermal contraction. These procedures are expected to require little, if any, increase in cost, and a dam constructed with RCC remains the most economical choice for the location.

For additional information contact Dennis Hopman, NPPEN-FM-C, Comm: 503/221-6455; FTS: 423-6455.

## CONVERTING FROM IN-HOUSE TO A/E CONTRACTING

John C. Kliethermes  
St. Paul District

The St. Paul District has significantly increased the A/E Contracting workload in the past year. The increase in contracting has had both positive and negative effects on milestones, quality of products, engineering capability, and morale within the branch. The CESEC presentation relates changes within the Engineering Division of the district, showing an increase in contracting load from \$500,000 to \$5 million within 3 years.

For additional information contact John C. Kliethermes, Design Branch, NCS, Comm: 612/725-7329; FTS: 725-7329.

## CLASSES



"FAST-TRACK" DESIGN/CONSTRUCTION OF THE CENTRAL  
COMMAND HEADQUARTERS, MACDILL AFB

Thomas L. Fultz  
Structural Engineer, Mobile District

The Central Command is a joint task force with the mission of rapidly responding to the United States defense interests in the Middle East. The command is headquartered at MacDill AFB, Florida. The Headquarters facilities were designed and constructed within an eight-month period using "Fast-Track" procedures.

"Fast-Track" design/construction procedures are used to expedite the completion of a project. The time savings from "Fast-Track" is developed by overlapping the design process and construction process.

The use of "Fast-Track" procedures on the Central Command facilities impacted the normal construction process in two general areas. The contract award process involved a source selection and the award of a letter contract. These projects were not built by the low bidder, but by contractors who knew how to expedite construction.

The second area of construction impact involved the compressed schedule. A good labor relationship with the contractor was necessary in order to achieve required productivity levels. All potential conflicts had to be resolved quickly. On-site representatives of the User's staff and of the Air Force Regional Civil Engineering Office coordinated all change orders. The Mobile District Office provided extra staff to expedite shop drawing review.

The design of the Central Command facilities was also impacted by the use of "Fast-Track" procedures. The building system selection was based upon the reduced construction time available and the local availability of materials. Cost was considered, but only to the extent it would not affect the schedule completion.

The Central Command facilities were divided into three phases. Phase one was the 75,000 Support Facility (SF) command and control area of the Headquarters building. Phase two was the 20,000 SF maintenance and warehouse area housed in the Support Facility. These two phases were completed in less than eight months. The third phase of the facility was a 65,000 SF addition to the

Headquarters building to house administration and logistic staff. This addition was constructed after phases one and two were complete.

The Headquarters building has a structural steel frame with a concrete second floor slab and a built-up roof system on a metal deck with bar-joist framing. The phase-one portion utilized a composite steel and concrete slab for the second floor. The Headquarters addition utilized a concrete slab on a metal deck with bar-joist framing. The change was based upon the contractor's input from the first phase.

The SF has a reinforced concrete frame. The roof utilized standing-seam metal on bar joists framed onto joist girders which were at 25-ft spacing. The joist girders span 99 ft 8 in. between 20- by 16-in. reinforced concrete columns.

The Mobile District learned many lessons from the Central Command "Fast-Track" projects. Primary among all lessons was that the driving force behind successful completion was the well-justified need for the expedited schedule. The critical schedule demanded a team effort from all concerned!

A "Fast-Track" design is driven by construction. The building system selection must be influenced by the local availability of materials and the compressed construction period. A valuable element of "Fast-Track" design is any input the construction contractor may have in the design process. Since construction begins simultaneously with design, well-defined functional criteria must be provided by the user. The initial letter contract for construction depends heavily on the user's design criteria.

Construction elements critical to the "Fast-Track" effort included the contractor's management of labor, materials, and equipment. The expedited schedule cannot be achieved without persistent hard work, proper material availability, and adequate equipment use. Other critical construction elements are prompt, shop-drawing review, careful equipment procurement for long lead-time items, and on-site representatives to resolve conflicts.

The Mobile District has successfully applied "Fast-Track" procedures. The expedited schedule of a "Fast-Track" project presents many challenges, but adequate leadership within the Corps of Engineers can meet such challenges.

For additional information contact Thomas L. Fultz, SAMEN-DT, Comm: 205/690-3489; FTS: 537-3489.

## BONNEVILLE NAVIGATION LOCK STRUCTURAL DESIGN

Norman Tolonen, Coordinator  
Columbia River Projects

Jerry Maurseth  
Supervisory Structural Engineer  
Portland District

A new navigation lock is proposed at the Bonneville Project on the Oregon side of the Columbia River. This presentation will briefly describe the various features of the proposed lock structure, then point out several unique design problems.

At the upstream approach to the lock, a floating guide wall (approximately 900 ft long) would be located on the north side. It would be supported by the lock intake monolith and a 55-ft diameter sheet-pile guard cell. At the downstream approach to the lock, two 55-ft diameter sheet-pile guide cells would be located where the approach channel intersects the Columbia River. Towboat and barge assembly impact loads must be considered in the design of floating guide wall and sheet-pile cells.

On the south side of the upstream approach, a buttress diaphragm guard wall would be constructed using the slurry trench construction method. This wall would function as a cutoff wall and retain the existing hillside where the Union Pacific Railroad would be relocated. The hillside is potentially unstable. The deflection of the buttress diaphragm wall must be determined accurately to size a wall member stiff enough to prevent excessive soil movement.

The upstream sill of the lock would consist of the existing rock formation. The columns of rock material would be rock bolted together to form a block monolith. A rock-bolting design procedure is required to insure that the assembly of rock columns functions as a solid block. The surfaces would be capped with concrete, forming a structure which would be effective for retaining the upper pool when the upstream lock gate is in its closed position.

The downstream wing wall and guard wall would be constructed of roller compacted concrete. This construction method allows for an optimum monolith shape to be quickly and easily formed.

The analysis and design methods employed for each of these structural features are explained for your use, as they may be applicable to various future projects.

For additional information contact Jerry Maurseth, NPPEN-DB-SA, Comm: 503/221-6568; FTS: 423-6568.

L&D 26R FIRST-STAGE DAM, DESIGN,  
AND CONSTRUCTION CASE HISTORY

Thomas J. Mudd  
Structural Engineering Section  
St. Louis District

This talk will describe the history of the formulation of design, analysis, preparation of specifications, and actual construction of the First-Stage 6-1/2 Gate Bays of the dam for Lock and Dam No. 26(R).

Construction techniques and problem solutions related to industry standards, inspection, construction sequencing, pile driving, and pile load tests will be described.

Lessons learned during the First-Stage Construction as incorporated in the Second-Stage Lock construction will be covered.

Finally, the overall project status for completion of the project will be covered.

For additional information contact Thomas Mudd, LMSED-DA, Comm:  
314/263-5524; FTS: 273-5524.

## L&D 26R LOCK, DESIGN, AND CONSTRUCTION SEQUENCE

Roger Hoell  
Structural Engineering Section  
St. Louis District

This presentation covers the evolution of the construction sequence for the Lock Cofferdam from the beginning of work on plans and specifications through the actual construction of the first portion of the cofferdam. This cofferdam is especially critical because of its location in an area of the Mississippi River that has become deeply scoured and because of the necessity to maintain river traffic past the construction area through an extremely restricted navigation pass. A detailed construction sequence evolved as a result of work on a CPM to determine a reasonable minimum contract duration for the Lock contract. This work necessitated a series of model studies of the construction sequence performed by the Waterways Experiment Station. These studies showed that the construction sequence was indeed critical, both from a constructibility and a navigation standpoint. The model tests also showed a need for nonstandard construction aids to build the cofferdam cells, such as local deflectors and temporary local scour protection.

The difficulties of dismantling the First-Stage Dam Cofferdam under two separate contracts while concurrently building the Second-Stage Lock Cofferdam are addressed. Compounding the situation is the fact that the sheetpiling removed from the dam cofferdam must be rehabilitated and reused on the lock cofferdam. In addition, the lock contractor has elected to construct the cells at the river bank, float them to their ultimate location, and then drive the cell as a unit. This will be the first application of this procedure in the United States.

For additional information contact Roger Hoell, LMS&D-DA, Comm: 314/263-5693; FTS: 273-5693.

INTEGRATED STRUCTURAL ENGINEERING SUPPORT FOR THE  
FEMA KEY WORKER BLAST-SHELTER PROGRAM

Paul M. LaHoud  
Structural Section  
Huntsville Division

The Federal Emergency Management Agency (FEMA) requested that USACE act as its engineering arm in developing a family of structures to protect "Key Workers" from nuclear weapons' effects. In support of this effort which involved elements at the division, laboratory, and district levels, a new and more cost-effective design procedure for shallow buried structures subjected to weapons effects was developed. This presentation discusses the program requirements, the structural engineering analysis and design procedures developed, and the interaction of the different USACE elements involved. The analysis procedure developed considers soil-structure interaction and in-plane compressive and tensile membrane forces as they enhance the resistance of shallow buried flat-roof structures subjected to blast overpressure. The new method results in structure designs which are substantially more economical than previous design methods would have provided.

For additional information contact Paul M. LaHoud, HNDED-CS, Comm: 205/895-5410; FTS: 873-5410.

DYNAMIC SOIL-STRUCTURE INTERACTION EFFECTS ON AND  
REINFORCEMENT DETAILS FOR BLAST-SHELTER DESIGN

Sam A. Kiger and Stanley C. Woodson  
Research Structural Engineers  
US Army Engineer Waterways Experiment Station

The Huntsville Division (HND) has been tasked by the Federal Emergency Management Agency to design a family of civil defense blast shelters known as Key Worker Blast Shelters. The Waterways Experiment Station is supporting the HND effort with design calculations and structure verification tests.

The objective was to develop an economical blast shelter to withstand a peak overpressure of at least 50 psi from a 1-Mt weapon. At this overpressure, protection from radiation would require that the structure be buried. Therefore, design calculations including our best understanding of dynamic soil-structure interaction (SSI) were used. Also, construction reinforcement details which economically enhance the blast resistance of a reinforced concrete structure were investigated.

The inclusion of dynamic SSI effects resulted in a much more efficient design. For example, assuming a 4-ft depth-of-burial and an 11-ft unsupported roof span, design calculations indicated a minimum roof thickness of 17 in. if SSI is ignored, and 9.25 in. thick if SSI effects are included in the calculations. The SSI calculations were evaluated with a series of 12 dynamic tests which simulated nuclear overpressures on 1/4-scale structural models. Tests were conducted at various depths of burial and in three different types of backfill with peak overpressures varying from about 50 to 150 psi.

The effects of stirrup details, principal reinforcement details, and roof/wall joint details on the load-response behavior of roof slabs were investigated. Twenty-eight reinforced concrete slabs and box elements were tested under uniform surface pressure either to failure or to deflections that exceeded 15 percent of clear span.

Tests results indicate that the design utilizing SSI effects will conservatively withstand the peak overpressure of 50 psi even in very poor (low-shear strength) backfill and will withstand a maximum overpressure of about 170 psi before total collapse. Construction details which resulted in a ductile failure mode with limited spalling were recommended. These include: (1) a principal reinforcement arrangement which provides the desired failure

mode in the absence of costly stirrups, and (2) a roof/wall connection detail that prevents premature roof collapse due to joint failure. These recommendations have been adopted and will be further evaluated in a prototype shelter scheduled for testing in June 1985.

For additional information contact Dr. Sam Kiger or Mr. Stanley Woodson, WESSS, Comm: 601/634-3696; FTS: 542-3696.

## CONCRETE CHANNELS, DRAINAGE AND FREEZE PROTECTION SYSTEM

John H. Plump  
Civil Engineer  
St. Paul District

The St. Paul District is currently developing plans and specifications for two projects which involve long concrete channels. An important consideration in the design is the control of water and the impacts of freezing. Numerous solutions have been suggested including relief holes, under-slab drains, insulation and frost-free material. The design of each of these, plus others, involves many variables and factors of risk. This presentation would discuss the design concepts, alternatives, and selected plan.

For additional information please contact John H. Plump, Civil Engineer, NCS&D-D, Comm: 612/725-7629; FTS: 725-7629.

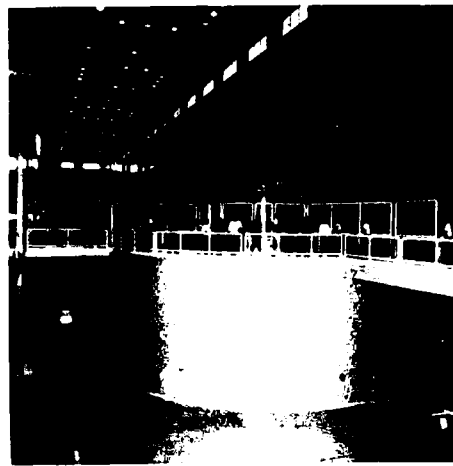
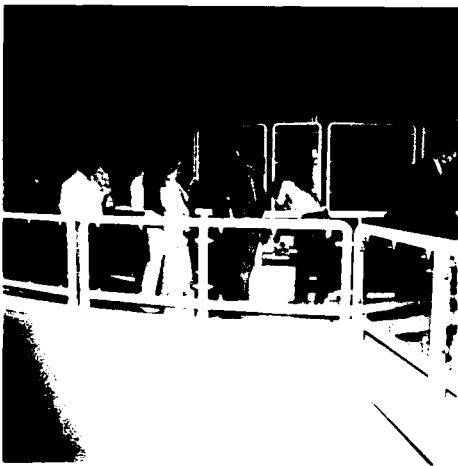
RESTORATION OF BUILDING 3001, TINKER AFB  
FAST-TRACK DESIGN/CONSTRUCT

Reggie Kikugawa  
Structural Section  
Tulsa District

Fire damage to 652,500 sq ft of a jet engine shop idled thousands of Air Force employees. Most of the structural damage was limited to the building's roof framing. All of the bar joists, X-bracing, and metal deck were destroyed. Damage to trusses and columns varied from minor to complete collapse. Safety required structural stabilization and asbestos insulation removal before reconstruction could begin. The high cost to the Air Force for each day the building was out of operation led to a fast-track design/construct schedule of seven months for the restoration.

For additional information please contact Reggie Kikugawa, Comm:  
918/581-7225; FTS: 745-7225.

## FIELD TRIP



PROBLEMS WITH LONG-TERM CONCRETE DEFLECTIONS  
IN A HOT, ARID CLIMATE

A. O. Werner, P.E.  
Chief, Structural Section  
Middle East Division

Environmental conditions encountered in a desert climate such as Saudi Arabia tend to amplify long-term deflection in concrete flexural members. The Corps' vast construction program in the Middle East has resulted in experiencing several high concrete deflection phenomena and the development of means to cope with it.

The projects in Saudi Arabia have included an \$8 billion military city that will house a population of 70,000 and the military academy of Saudi Arabia, which in itself is a small city housing about 20,000 people. In addition, we have designed and built port facilities, many military headquarters, and hardened command-center facilities. Of all this construction, which totals around \$20 billion, almost all of the structures, with only a few exceptions, are made of concrete. The structures are divided somewhat evenly between cast-in-place and precast, mildly reinforced construction, with a smaller share going to prestressed construction. There have been very few structural problems with the finished construction; however, we have had more than our share of one particular phenomenon, a high, long-term deflection of concrete members that sometimes reaches three to four times more than would be expected in a moderate climate.

There are four broad factors that affect long-term deflections:

- a. Materials. A low rigidity of the coarse aggregate produces a low modulus of elasticity of the concrete which results in larger deflections at all ages. Also, the cement content can affect the creep value.
- b. Design. Creep is proportional to stress up to about 40 percent of the strength of the member--above that, the rate of creep increases significantly. The best candidates for high, long-term deflections are highly stressed, relatively thin beams with minimal steel in the compression zone.
- c. Construction. Creep can be affected by construction deficiencies such as an improper concrete mix, misplacing reinforcing steel, or improper curing.
- d. Environment. The concerns due to the previous three factors are pretty consistent, regardless of the area of construction, while the

environmental factors of a hot, arid climate like Saudi Arabia increase the potential for creep over construction in most areas of the United States.

Creep is known to increase at:

- a. High temperatures.
- b. Changing temperatures.
- c. Low humidity.
- d. Changing humidity.

We have highlighted the need to consider long-term concrete deflections in our A-E and in-house design-criteria documents. These precautions and considerations are the same as for designs for structures anywhere except, of course, more emphasis needs to be placed on them in a hot, arid environment.

We encourage the use of the deepest feasible in flexural members and avoid thin, highly stressed sections. This, also, usually increases the ratio of concrete volume to its surface dimensions which lowers the creep factor. Secondly, the use of compressive reinforcement can significantly reduce long-term deflections. The use of compression steel tends to offset the movement of the neutral axis caused by creep as well as movement caused by progressive cracking in the tensile zone. Finally, for long span structure, consideration is given for increasing camber in anticipation of larger creep values.

For additional information contact A. O. Werner, MEDED-TS, Comm: 703/665-3775; FTS: 652-3775.

UNIQUE FACTORS INFLUENCING THE DESIGN AND CONSTRUCTION  
OF THE FORT CAMPBELL HOSPITAL, FORT CAMPBELL, KY

Daniel L. Parrott  
Project Engineering & Configuration Section  
Mobile District

The Fort Campbell Hospital, located outside of Clarksville, Tenn., is a multipurpose complex serving the needs of over 30,000 soldiers and their dependents. Housed in four separate buildings totaling over 464,000 sq ft, this hospital complex was designed and built to meet unusual design criteria and loading conditions.

The Fort Campbell area is located in a region of widespread Karst topography. This is indicated by the numerous settlements, sinkholes, and a general lack of surface drainage in the area. To avoid the possibility of settlement or sinkhole development beneath the complex, major steps were undertaken both before and during construction. Most important of these steps was the grouting of the foundation.

The grouting program was initiated to help seal off the top of the highly permeable limestone bedrock. Over 272,000 cu ft of grout was pumped into over 44,000 linear ft of grout holes. The total cost of the grouting program was \$1,500,000. Today, after over three years of operation, no appreciable settlement has been noted.

This project site is located approximately 120 miles from the assumed epicenter of the New Madrid Earthquakes. Because of its proximity, stringent seismic operational criteria were employed. Two of the buildings, Building "B," the Diagnostic/Treatment Center, and Building "D," the Mechanical/Boiler Plant building, were designed to be fully operational within six hours after a design earthquake. At larger ground motions, these buildings are designed to avoid a total collapse. The other two structures, Building "A," the Bed Tower, and Building "C," the Outpatient Clinic, were not considered essential enough to warrant the extra seismic protection, and are designed against collapse.

Buildings "B" and "D" were designed using a time-step dynamic analysis, based on the El Centro earthquake of 1956. Buildings "A" and "C" were designed according to the 1974 Structural Engineers Association of California code as contained in the TM-5-809-10.

Each building was structurally designed with consideration given to the ultimate function of the building. Building "A" is designed as a reinforced concrete shear-wall structure on a 5-ft thick reinforced concrete foundation mat. Buildings "B" and "C" are designed to carry the lateral loads from the floors to structural steel braced frames, then into an array of shear walls at the first floor level. Building "B" utilizes a reinforced grid mat foundation, and Building "C" uses spread footings on compacted fill. Building "D" utilizes perimeter shear walls to resist seismic loads, which are then attached to the reinforced concrete mat foundation. Buildings "A," "B," and "D" are designed to span over a 25-ft 0-in. diameter sinkhole, if one should develop beneath the structure.

Buildings "B" and "C" were separated to allow a savings in construction costs. Because no litter-borne patients are treated in the outpatient clinic, Building "C" was classified for business occupancy, permitting the use of more economical building materials.

The start of construction was during September 1977. The end of construction was in December 1981. The final cost for the hospital complex was \$56,000,000.

In 1984, the Col. Blanchfield Army Hospital at Fort Campbell, Ky. won the Department of Defense Architectural Design Award for Medical Facilities.

For additional information contact Daniel L. Parrott, SAFEN-DC, Comm: 205/694-3695; FTS: 537-3695.

THERMAL AND STRESS ANALYSIS OF LONGITUDINAL  
JOINTS FOR THREE GORGES DAM, CHINA

David A. Dollar  
Civil Engineer  
Bureau of Reclamation

The Yangtze River is the third largest river in the world. The average annual flow is  $453 \times 10^9 \text{ m}^3/\text{s}$  ( $367.3 \text{ by } 10^6 \text{ acre-ft}$  or an equivalent discharge of  $507,280 \text{ ft}^3/\text{s}$ ). The construction of a dam at the Three Gorges site will provide flood control, power, and navigation. The YVPO (Yangtze Valley Planning Office) of the Peoples' Republic of China has decided on the construction of a 165m (541.2 ft) high concrete gravity dam with a crest length not including the navigational facilities of 2,330 m (7642.4 ft) and a total volume of  $14.5 \times 10^6 \text{ m}^3$  ( $18.95 \text{ by } 10^6 \text{ yd}^3$ ). The majority of the dam will be very close to the maximum section in size and shape.

Many problems, both in design and construction, will arise from the magnitude of this project. One such problem is the tensile stresses which arise from the temperature increase due to the heat of hydration of cement in mass concrete placements, the subsequent cooling to the final stable temperature by embedded cooling coils, and the restraint provided by the foundation. A key concept in this process is that the concrete modulus of elasticity increases with time.

In order to determine the effects of this condition, both USBR Engineering Monograph No. 34 "Control of Cracking in Mass Concrete Structures," and the finite-element method were used. Studies of zero, one, two, and three longitudinal contraction joints in the 406-ft-wide base show that the temperature rise from heat of hydration is a maximum of  $33^\circ \text{ F}$  for all block lengths and that the thermal stresses decrease as the block length decreases. The results indicate that, assuming homogeneous linearly elastic concrete and foundation rock, tensile stresses will exist which are very close to the expected tensile strength even with blocks only 101.5 ft long.

For additional information contact David A. Dollar, US Bureau of Reclamation, Comm: 303/236-4005; FTS: 776-4405.

SAVINGS THROUGH ENGINEERING ON THE  
DOWNSTREAM GUIDEWALL AT L&D 26(R)

John Jaeger  
Structural Engineering Section  
St. Louis District

The FDM Design On Lock And Dam 26(R) Downstream Guidewall was a conventional laisson-founded design. The wall would be constructed in the dry by constructing a cofferdam around the guidewall. Twenty-one ft of sand fill would then be deposited within the cofferdam with a twenty-four-in. thick concrete slab tremied on top of this fill as a working platform. The construction area within the cofferdam would then be dewatered. The wall would then be constructed in the dry by conventional methods. As the FDM Design progressed, it was determined that the number of laissons would increase by fifty percent in order to develop adequate laisson capacity. It was also determined that the cost associated with installing the cofferdam, including the sand fill, tremied concrete working slab, and dewatering was estimated to cost 9.2 million dollars.

An engineering study was conducted to determine the most feasible method to construct the guidewall without dewatering the site. It was determined that a pile-founded tremie concrete filled sheet-pile cell with precast concrete beams could be constructed in the wet without dewatering the site. The design change was approved by OCE resulting in an estimated construction saving of 7.9 million dollars. The presentation will discuss the type of design and construction required to construct the sheet-pile cells and precast concrete beams.

For additional information contact John Jaeger, LMSFD-DA, Comm: 314/263-5856; FTS: 273-5856.

## UNDERGROUND MUNITIONS STORAGE FACILITIES STUDY

Dr. James D. Prendergast  
Engineering and Materials Division  
Construction Engineering Research Laboratory

For more than 100 years, igloos or similar types of aboveground structures have been used for storing ammunition and weapon systems. The spatial relationships among a group of these igloos are governed, for safety, by the type and quantity of the stored contents to ensure that an accidental explosion in one unit does not cause sympathetic detonation of the contents of an adjacent unit. Consequently, most igloo storage areas require large tracts of real estate. Since these areas will not contain an accidental explosion, the material must be vented to the environment. Moreover, since the igloos are aboveground, all munitions transfer activities are subject to public surveillance and terrorist attack. Physical security for these storage areas requires a large security force, and electronic surveillance to protect large areas is expensive. Working with the Air Force, and in conjunction with the Defense Nuclear Agency and the Department of Defense Explosive Safety Board on relocation of a major Air Force storage facility, USA-CERL developed an innovative underground munitions storage concept which overcomes these problems. This concept enabled the current aboveground munitions storage area to be relocated into a new hardened underground munitions storage complex with collocated maintenance, administrative, and loading dock facilities. The new concept provides absolute environmental protection and denies terrorist or dedicated commando unit's entry into the facility for a reasonable time period, regardless of their means of attack. The initial costs for the aboveground Air Force igloo and the adopted underground concepts were the same. However, the annual cost savings represented by the allowable reductions in security force will enable the estimated \$50 M construction cost to be amortized in about 5 years. This facility is currently under final design by MRO and is scheduled for completion this fiscal year. This innovative storage concept has been applied to storage requirements for the Navy and Army, as well as the Air Force. The Navy project is in the initial stages of design and requirements are being finalized for the Army project. Currently, USA-CERL and MRO are working jointly to capture and integrate the design

experience and lessons learned from the Air Force storage facility into the final concept and design requirements for the Army's storage facility demonstration project.

For further information contact Dr. James Prendergast, USA-CERL-EM,  
Comm: 217/373-7248; FTS: 958-7248.

## UNDERGROUND MUNITIONS STORAGE COMPLEX, DESIGN

William H. Gaube  
Chief, Hardened Structures Design Section  
Special Projects Branch, Engineering Division  
Omaha District

The Underground Munitions Storage Complex consists of a fully buried munitions storage and maintenance building and surface facilities which include a Squadron Operations building, Utilities building, and supporting roads and parking. The design of the buried storage and maintenance building was augmented by quarter-scale model tests conducted by the Waterways Experiment Station (WES). These tests covered both internal and external threats.

The internal threat includes an accidental explosion of munitions during transport, maintenance, or storage. All explosive gas products from such an explosion must be contained within the building. Model studies conducted by WES have shown that containment of an internal blast is feasible.

The external threat involves an attack by a terrorist team to gain force entry. The attack scenarios include an attempt to excavate the earth cover, penetrate the roof slab, and an attempt to ram the entrance with a vehicle.

For additional information contact Mr. William H. Gaube, MROED-SH, Comm: 402/22-4918; FTS: 864-4918. Additional information on the model studies can be found in WES Technical Report SL-84-14, Kirtland Underground Munition Storage Complex Model Designs, Construction, and Test Data, September 1984; WES Technical Report SL-84-16, Vulnerability of an Underground Weapon Storage Facility, September 1984; and WES Technical Report SL-85-1, Advanced Storage Concepts; Roof Entry Tests, February 1985.

## A COMPREHENSIVE APPROACH TO DAM SAFETY

James R. Jordan  
Structural Engineer  
Mobile District

Dam Safety is the "Umbrella of Management" over the programs of Technical Assistance to local authorities, Periodic Inspection of our own projects, Phase I Inspection of Non-Federal Dams, Dam Safety Plans and Training for our project personnel, Emergency Drawdown of Dams in Danger of Imminent Failure and the Inventory of Dams.

The authorities exist for this approach and stem from the appointment of a Dam Safety Officer at each echelon of the Corps' decentralized organizational structure. Mobile District supports an overall policy of Dam Safety Management.

The Periodic Inspection Program (PICES) at Mobile District is a detailed, technical inspection involving project supervisors and office engineers, substantiated by instrumentation and documented as a historical record of the status of the project. The Dam Safety Training for Operations Personnel (DSTOP) is directed toward the working level personnel and prepares them what to watch for, when to take emergency action, how to react to emergency situations, and whom to notify. The District responds with technical assistance and emergency drawdown equipment if requested by the Governor when dams are in imminent danger of failure. The District retains the Inventory of Non-Federal Dams and the records from the Phase I Inspection of these dams. The District supports legislation for a State-wide permitting, inspection, and management program of Alabama's dams.

Dam Safety is a high visibility subject from a social, political, and technical aspect. Command emphasis from the Secretary of the Army through the Chief of Engineers to District Engineers has identified Dam Safety as one of the emerging national needs requiring a coordinated and comprehensive management approach.

For further information contact Jim Jordan, SAMEN-DN, Comm:  
205/690-2634; FTS: 537-2634.

## NORFORK DAM STRESS ANALYSIS

David A. Hill  
Little Rock District

Norfolk Dam is a conventional mass concrete gravity structure located in northern Arkansas. Before construction was completed, extensive cracking appeared in all monoliths with a major vertical crack in the midsection of monolith 16 which extended approximately 7/9 of the dam height.

In 1962 Dr. R. W. Clough was hired to analyze the structure. Using the original design flood condition or basis for his study, he concluded that the dam was safe. A new analysis was necessitated due to the changes in probable maximum flood conditions. The finite-element method was utilized for the current structure investigation and attempts were made to refine the earlier investigation.

The program GTSTRU DL was selected for the analysis and was run on a Control Data Mainframe computer. Two-dimensional triangular and quadrilateral elements were used to construct the finite-element model.

Loading consisted of the weight of the structure itself, while hydrostatic (lateral and uplift pressures) and temperature loadings were due to seasonal temperature variation.

This study showed clearly that, although several rather significant stress concentrations might be associated with the crack, the magnitude of the total stresses were not excessive. It was then concluded that the dam is safe under the hydrostatic loadings derived from the probable maximum flood conditions and validated this same conclusion drawn from the earlier study.

For additional information contact David A. Hill, Comm: 501/378-5266;  
FTS: 740-5266.

STRUCTURAL BEHAVIOR OF MITER GATES,  
AN INTERPRETATION OF RECENT FINITE-ELEMENT STUDIES

Joseph P. Hartman, P.E.  
US Army Engineers Division, Southwestern

The CASE Task Group on Miter Gates has sponsored several finite-element studies of miter gates. A thorough evaluation of the results will be published later this year. This presentation is a summary of these studies.

The studies addressed behavior of a conventional horizontally framed miter gate. Results can be summarized as follows. Most conventional assumptions have been verified: the girders behave as segments of a three-hinged arch; temperature-induced stresses are small and localized and may be safely ignored; stresses in vertical diaphragms are small except in areas near a concentrated applied force; torsional behavior is dictated by the presence of diagonals, which permit closed-section torsion rather than thin-member torsion. Though the studies also provided information on the distribution of concentrated loads into the surrounding gate elements, this is not covered in current design guidance. Buckling behavior of a typical girder was investigated. Adequate stiffeners must be provided to prevent local buckling of thin members, but overall buckling of the girder does not seem to be a serious possibility.

For additional information please contact Joseph P. Hartman, Comm: 214/767-2397; FTS: 729-2397.

STRUCTURAL BEHAVIOR OF ALTERNATE  
CONFIGURATIONS OF MITER GATES

Michael D. Nelson, P.E.  
Project Manager  
Seattle District

In the last few years there has been an increased interest in miter gate leaf stiffness, or torsional resistance. As gate heights increase, the need for additional torsional resistance to dead load and temporal or surge loads becomes a significant design consideration.

The Computer-Aided Structural Engineering (CASE) Task Group on Miter Gates has been investigating alternate methods of controlling the torsional stiffness of large gates. Finite-element computer model studies have been performed to evaluate the stiffness of gates now in operation in the Corps of Engineers Districts, Mobile and Huntington. The task group has studied the possibility of eliminating the conventional diagonal stiffeners from gates and replacing them with skin plate on a portion the downstream face of the gate. These alternate configuration or "torque tube" models provide zones of greatly increased torsional stiffness in localized regions of the gate.

The torque-tube models appear to offer an excellent alternative to conventionally stiffened gate design, particularly on gates over 100 ft tall. These large gates will require increased stiffening beyond what conventional diagonals can now provide.

For additional information contact Michael D. Nelson, NPSEN-PL-CP, Comm: 206/764-3458; FTS: 399-3458.

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EM-111-2-2703, Lock Gates And Operating Equipment.  
US Army Engineer Waterways Experiment Station. 1985. Technical Report  
K-85-X, Analysis of John Hollis Bankhead Lower Miter Gate, Draft Report,  
Vicksburg, Mississippi.

## CERRILLOS DAM OUTLET WORKS

G. R. Navidi  
Chief, Structures Section  
Jacksonville District

The Cerrillos Dam is part of the Portuguese and Bucana Rivers (P&B) Project, located near the city of Ponce in the southcentral part of Puerto Rico. Authorized by Congress in 1970, for the purposes of flood control, water supply, and recreation, the P&B Project consists of three parts: the Cerrillos Dam and Reservoir, the Portuguese Dam and Reservoir, and the downstream channel improvements for both dams. Construction on the P&B Project began in 1975 by awarding the contract for the first section of the channel improvements. The Cerrillos Dam is currently under construction with a scheduled completion date of 1990, and the Portuguese Dam construction is expected to begin in 1988.

The Cerrillos Dam will be a 320-ft-high, zoned rockfill dam with an ungated, 400-ft-wide spillway at the right abutment and an outlet works at the left abutment. The outlet works consists of an inclined intake/control structure and an 18-ft-diameter R.O./diversion tunnel with stilling basin. The inclined structure is 436 ft long and will be constructed along a 48-deg incline against the left abutment. The concrete lined tunnel, completed in 1984 under an early contract, is 18 ft in diameter, 1,425 ft long. The difficult site and highly unpredictable climatic conditions necessitated special consideration in developing the design and sequence of construction of the outlet works. The narrow dam site dictated the use of a dual function (diversion/ R.O.) intake structure and tunnel. This concept for handling the river diversion requirements resulted in a considerable cost savings by eliminating the need for a separate diversion tunnel. Because of the seismicity of the dam site, (Zone III), an inclined intake/control structure was selected in lieu of a conventional freestanding structure which would have measured 360 ft high from foundation. The unique configuration of the structure posed unusual design considerations, such as single versus multiple monolithic construction, bulkhead design, interior layout and design, and foundation stability along the slope.

A limited number of feature design memoranda is available upon request. For additional information contact Ray Navidi, SAJEN-DS, Comm: 904/791-2206; FTS: 946-2206.

## MUD MOUNTAIN DAM INTAKE TOWER ANALYSIS

Paul C. Noyes, P.E.  
Structural Design Section  
Seattle District

An earthquake analysis was made of the intake tower at Mud Mountain Dam, on the White River, in the state of Washington. The dam is earth and rock fill and the tower is a concrete structure built against a sloping rock face. Construction was completed in 1948. The tower is generally circular in section: the back half, a solid concrete shell, and the front half, a row of concrete columns which form a trashrack. Regulating equipment for controlling reservoir discharge is not contained within the tower. The tower is founded at elevation 270. It is built against the rock to elevation 1,015. The remaining tower is freestanding and tops out at elevation 1,072 with the spillway crest at elevation 1,250.

Chopra's simplified method of dynamic analysis was used for the free-standing portion. For the portion of the tower against the rock face, the seismic coefficient method was used. Hydrodynamic loads were determined using Westergaard's method. The Seed and Lysmer response spectrum for rock used was scaled to 0.35g. The actual strength of concrete was not known, but was assumed to be 3,000 psi. It was assumed that there was no bonding between the concrete and rock. An equivalent area of concrete was added to the section to account for the reinforcing. Ratio of steel to concrete was only .002.

Bending stresses were calculated for the base of the tower. Maximum tensile stress in concrete was 3,000 psi. Allowable tensile stress is taken to be 375 psi. Because the tower was shown to be so highly overstressed, a more sophisticated method of analysis was not warranted. It was concluded that the tower would fail under the design earthquake.

For additional information contact Paul C. Noyes, P.E., NPSEN-DB-ST,  
Comm: 206/764-3791; FTS: 399-3791.

CONCRETE REACTIVITY PROBLEMS AND REMEDIAL  
MEASURES AT TVA PROJECTS

Harold C. Buttrey  
Civil Project Engineer  
Tennessee Valley Authority

The most important reaction creating concrete growth is that between minor alkali ( $\text{Na}_2\text{O}$  and  $\text{K}_2\text{O}$ ) hydroxides from cement and the concrete aggregates. Two distinctly harmful reactions have been recognized: the alkali-silicate and alkali-carbonate reactions. These alkali-aggregate reactions are complex and not completely defined. Concrete deteriorating from an alkali-aggregate reaction, regardless of the type, develops an obvious network of cracks called pattern or map cracking. Expansive forces resulting from an alkali-aggregate reaction may also cause structural cracking and excessive movements, which can threaten the integrity of a structure.

Structural problems due both to alkali-silicate and alkali-carbonated reactivity in the concrete have surfaced at TVA's Fontana Dam, structural cracking was discovered in 1972 in the walls of the foundation drainage gallery in the curved portion of the dam near the left abutment. Further field investigations revealed the cracking to be extensive, resulting in a comprehensive program of analysis and repair. The repairs included grouting and posttensioning of the crack, and cutting a 100-ft-deep slot to relieve the longitudinal stress. Structural crackling at Hiwassee project has not, at present, required repair. A program of close surveillance and monitoring, however, is maintained at this project.

Most harmful reactions can now be prevented in proposed structures by interpreting the results of standard test methods. It is not possible, however, in existing structures to determine how far the growth phenomenon has progressed, how long the effects will have to be dealt with, or what the future effects will be. A program of close surveillance and monitoring is maintained at these projects, and problems are dealt with as they arise.

For additional information contact Harold Buttrey, TVA, Comm:  
615/632-3336; FTS: 856-3336.

CONCRETE REACTIVITY PROBLEMS AND REMEDIAL  
MEASURES AT CENTER HILL DAM

Ken Hull  
Civil Engineer  
Nashville District

Center Hill Dam is a rolled embankment and concrete gravity-type structure located about 70 miles southeast of Nashville, Tenn. Construction of the combination flood control and hydro-power project was begun in 1942 and completed in 1951.

Signs of minor distress in the spillway section have been evident for several years. Recent behavior, including binding of a tainter gate, the appearance of map cracking, and closing of some roadway expansion joints, indicated movement possibly due to volume change of the concrete. Since carbonate aggregate had been used in the concrete of this structure, the possibility of an expansive alkali-carbonate rock reaction was investigated.

Examination and laboratory testing of concrete cores and rock samples from the original quarry site verified that some of the aggregate used in the project was reactive. In-site concrete stresses in the seemingly affected areas of the dam indicated relatively high longitudinal stresses which were not thermal related. It has been concluded that all or part of the cracking in this structure is due to a deleterious alkali-carbonate rock reaction.

Though the structural integrity of the project has not been adversely affected, remedial measures necessitated by the "concrete growth" involved freeing the roadway expansion joints and modifying two tainter gates and their side seal plates. Inclinoimeters and very precise joint-movement devices have been installed at selected areas of the structure to monitor future movements related to the concrete reactivity.

For additional information contact Ken Hull, ORNED-D, Comm: 615/251-5617; FTS: 852-5617.

CAISSON FOUNDATION DESIGN AND CONSTRUCTION  
AT SAVANNAH RIVER PLANT

Kirti S. Joshi, P.E.  
Structural Engineer  
Savannah District

The Savannah River Plant is one of our country's few nuclear reprocessing plants. When plans were being made to bring an additional reactor on line, environmentalists expressed concerns regarding the relatively high temperature of the discharge water from the plant into the Savannah River. A cooling reservoir and dam was proposed which would hold the discharged water till the temperature of the water was within the required limits. Caissons were proposed for the foundation of the Intake Tower and under the pier of the Access Bridge. The structural foundation was underlain by 60 ft of very loose sand (SPT of 5 to 7), which in turn was followed by 20 ft of soft carbonaceous rock and mudstone called marl. The site was less than ideal for a dam but other concerns dictated that only this site could be used. Accordingly, an innovative design approach was required.

Design:

The final calculations for the Access Bridge and the Intake Tower showed that each caisson had to resist vertical load of 749 k and a seismic lateral load of 90 k. Soil interaction program COM 624G, "Laterally loaded pile analysis," was used to compute the size of the caissons and the reinforcement required. Modeling of the soil around the caisson was complicated since seismic activity would induce liquefaction of soil and consequently the caissons would lose lateral support from the ground line to a depth of 60 ft. A compromise was reached in modeling very soft clay in lieu of loose sand since it was argued that liquefaction of the sand would offer some limited amount of lateral support. It was agreed that a 60-ft layer of very soft clay would provide such a lateral support with initial subgrade modulus of 25 psi. Also, it was assumed that there would be no frictional resistance offered from the ground line to the bottom of carbonaceous rock. COM 624G showed that a 48-in. diameter caisson with approximately 4 percent reinforcement would be adequate to resist the seismic load. The fixity of the caissons with the above conditions would be located at depth of 42 ft.

The caisson was also designed to provide axial resistance of 749 k with factor of safety of 2.0. The total length of the caisson was 95 ft and was embedded 15 ft into the marl. Most of the axial load was resisted by end bearing.

Construction:

J. A. Jones of Charlotte, N.C. has the general contractor and the caisson installation was sub-contracted to CASE International of Chicago. CASE International brought in a crane mounted Rig--Hughes CLLDH "Super Duty" with maximum torque of 102,000 ft/lb and crowd of 8,000 lbs. The subcontractor had no problem drilling through the sand and the carbonaceous rock using a flight auger. Bentonite slurry was used to help prevent collapse of the hole. An 80-ft thick casing was inserted into the drilled hole and the last 3 ft was twisted inside the marl. The bentonite slurry was pumped out and a 48-in. diameter core barrell was used to drill the last 15 ft of the caisson. After cleaning the bottom of the hole and placing a reinforcement cage, concrete was pumped. The concrete mix had 7- to 9-in. slump. The reinforcement cage had experienced some dragdown on some of the longitudinal bars.

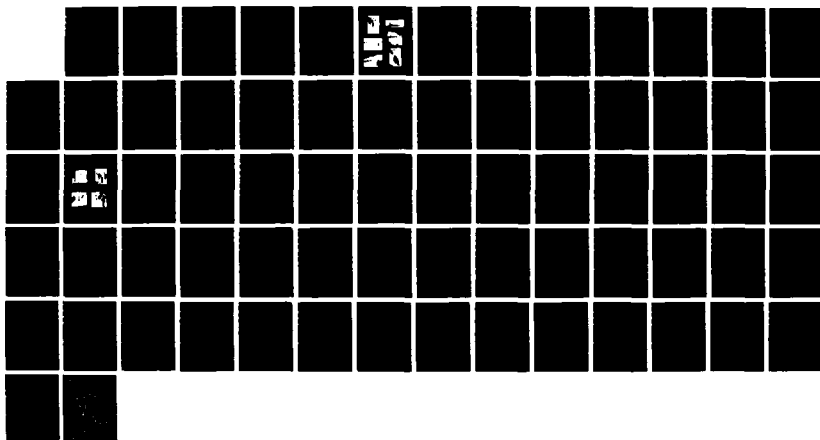
This project offers a novel approach since design and construction of deep caisson foundations are not ordinary run-of-the-mill projects which structural engineers come across. The presentation illustrates solutions to problems associated with specification writing, approximate size of drilling rigs to use, concrete dragdown forces, lifting of reinforcement cages, etc.

For additional information contact Kirti Joshi, SASSEN-DS, Comm: 912/944-5568; FTS: 248-5568.

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COMPLETION REPORT ON THE CORPS OF ENGINEERS STRUCTURAL 2/2  
ENGINEERING CONFER. (U) OFFICE OF THE CHIEF OF  
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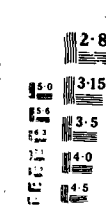
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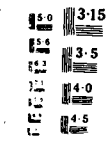
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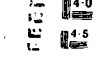
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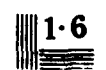
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## L&D 25 - GUIDEWALL REPAIRS AND STABILIZATION

Thomas R. Ruf and Thomas J. Leicht  
Structural Engineers  
St. Louis District

Lock and Dam 25 is the 27th of 29 locks and dams on the Mississippi River. It is located about 30 miles northwest of St. Louis, Mo. The project was completed in 1936. The guidewalls are located at the ends of the lock and are used by tows to guide into and out of the lock. The guidewalls consist of unreinforced concrete monoliths founded on timber piling.

The first part of the presentation discusses the repairs to the lower guidewall. The end monoliths of the lower guidewall were damaged by the prop wash and impact of tows landing on the end monolith. Instrumentation readings indicated the end monolith had deflected downstream approximately 4.5 in.

Several methods for repair were evaluated. The method selected to stabilize the lower guidewall was to cut a trench in the top of the end monoliths and install prestressing bars in the trench to tie the monoliths together. The trench was then filled with concrete. A sheet-pile cofferdam cell was constructed downstream of the lower guidewall to protect it from the impact of tows going upstream. This cell had to be designed to resist splitting caused by barge impact, along with meeting standard sheet-pile cell design criteria.

The second part of the presentation discusses the repairs to the upper guidewall. The problem with the upper guidewall appeared when a portion of the foundation material beneath the wall washed out due to the prop wash of tows leaving the lock and pushing upstream along the wall. A large hole developed in the fill material behind the guidewall when this material collapsed into the void beneath the wall. Instrumentation readings indicated the wall had deflected approximately one in. toward the river due to this void.

The upper guidewall was stabilized by tying it back with prestressing bars to a sheet-pile wall which was driven approximately 60 ft behind the upper guidewall. The bars were grouted into horizontal holes drilled in the back of the guidewall. Sufficient prestressing force had to be applied to mobilize the passive resistance of the sheet-pile wall, without overstressing the guidewall foundation piling. The void that had formed beneath the wall was filled by pumping a sand and water mixture through holes drilled

vertically through the wall. The repair work to the upper guidewall was all done without any interruption to river traffic.

For additional information contact Thomas Ruf and Thomas Leicht, Comm: 314/263-5693; FTS: 273-5693.

MAJOR REHABILITATION OF BOURNE AND SAGAMORE HIGHWAY BRIDGES,  
CAPE COD CANAL, BOURNE, MASSACHUSETTS

David R. Descoteaux  
Structural Engineer  
New England Division

Opened to traffic in June 1935, the Bourne and Sagamore Bridges are links in major state highways crossing the Cape Cod Canal at Bourne, Massachusetts. By the mid-1970's progressive deterioration due to the salt atmosphere and the effects of deicing salts used on the roadway during the winter had reached the stage where a major rehabilitation was necessary. This work was performed from 1980 to 1983 and included the following: replacement of roadway decks, miscellaneous steel repairs, repaving, repainting, replacement of hanger cables, and installation of a suicide deterring fence.

A basic consideration for the rehabilitation work was that interference with vehicular traffic be kept to an absolute minimum because of the importance of the bridges to the essential services of the community and their importance to the economy of the region. The major obstacle to this objective was the replacement of the roadway decks. This was accomplished by devising a construction sequence which maintained one lane open to traffic at all times and by installing a concrete-filled steel grid for the new roadway deck. It is estimated that the installation of the steel grid, in lieu of an in-kind replacement of the replacement of the original reinforced concrete desk, reduced construction time by one year.

Also, a unique feature was incorporated into the rehabilitation work to address the problem of suicides from the bridges. A suicide deterring fence, which extends 11 ft 9 in. above the bridge sidewalks and consists of 1-in. diameter steel palings at 6 in. on center, was installed. To date, the fence has performed its design function.

For additional information contact David R. Descoteaux, NEDED-DG, Comm: 617/647-8024; FTS: 839-7204.

## CORROSION PROTECTION OF PROJECTS IN LOUISVILLE DISTRICT

Ralph B. Snowberger  
Structural Engineer  
Louisville District

Protection has varied from project to project, but a pattern has developed in the locks, dams, and outlet works.

The coal-tar epoxy paint provides a long period of protection.

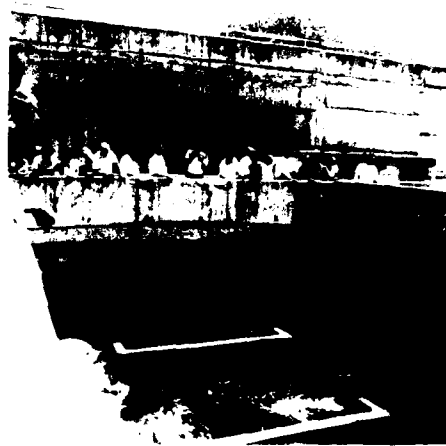
The vinyl paint systems now being specified provide a high degree of protection, but some problems have developed for maintenance painting at existing projects. These problems are: (a) adequately specifying the surfaces to be painted, (b) adequately specifying the surface preparation, (c) coordination of application procedures, and (d) ensuring paint and paint additive quality.

Cathodic protection protects underwater and underground surfaces after a paint failure until repainting takes place. The degree of protection required depends upon rate of corrosion anticipated and effect on the integrity of the structure.

The type of protection system depends upon location. Impressed current systems for lock gates have generally become inoperative due to normal wear and tear. Galvanic systems have had very little deterioration of the anodes.

For additional information please contact Ralph B. Snowberger, ORLED-T, Comm: 513/684-2152; FTS: 684-2152.

## FIELD TRIP



ILLINOIS WATERWAY, LOCKPORT, AND BRANDON ROAD  
LOCKS 1984 REHABILITATION

Denny A. Lundberg  
Project Manager  
Rock Island District

Lockport Lock is located at river mile 291 on the Illinois Waterway, immediately west of the city of Lockport, Ill. The lock is 600 ft long by 110 ft wide and has a lift of 39 ft. The lock is operated by a submersible vertical lift gate (with spare) at the upper end and a miter gate at the lower end. The filling and emptying system is the wall port type. The lock was completed in 1933 and became part of the Illinois Waterway System.

The design for major rehabilitation of Lockport Lock was presented in a General Design Memorandum dated May 1982. The rehabilitation project will be accomplished under four separate contracts totaling \$18.7 million. The first contract was awarded in September 1983 at a cost of \$8 million and included all rehabilitation requiring a complete lock closure. The lock closure was scheduled for 65 days between 5 July 1984 and 8 September 1984. The remaining contracts include work that will have minimum impacts to navigation. Project completion is scheduled for September 1986.

The Brandon Road Lock is located 5 miles downstream of Lockport Lock at river mile 286 in the city of Joliet, Ill. The lock is 110 ft wide by 600 ft long with a normal lift of 34 ft. The lock is operated by two sets of miter gates at the upper end and one set at the lower end. The filling and emptying system is the wall port type. The lock was completed in 1933 and also became part of the Illinois Waterway System.

The design for major rehabilitation of Brandon Road Lock and Dam was presented in a General Design Memorandum dated April 1983. The rehabilitation project will be accomplished under four separate contracts totaling \$22.3 million. The first contract was awarded in February 1984 and included all rehabilitation requiring a complete lock closure. This work was scheduled concurrent with the 65 day closure scheduled for Lockport Lock. The remaining contracts include work that will have minimum impacts to navigation on the lock and dam. Project completion is scheduled for June 1987.

The major features of design for both these projects included stability of elements of the existing structures, evaluation of the existing concrete,

concrete removal and replacement techniques, new miter gates at Lockport Lock, and rehabilitation of existing miter gates at Brandon Road. The presentation will cover the above design considerations along with solutions to problems encountered during construction.

For additional information contact Denny Lundberg, NCRED-DM, Comm: 309/788-6361, ext. 507; or 1/800/843-7582.

INVESTIGATION AND REPAIR OF JOHN DAY NAVIGATION  
LOCK DOWNSTREAM LIFT GATE

William Wheeler  
Structural Engineer  
Portland District

John Day Lock and Dam is located on the Columbia River about 110 miles upstream from Portland, Oregon. The major functions of the project are: power, navigation, flood control, irrigation, and recreation. The navigation lock is 657 ft long, 86 ft wide, and provides a maximum lift of 113 ft. The downstream lock gate is a vertical lift gate 88 ft wide and 113 ft high. It is a steel-welded structure consisting of 24 horizontal tied arches so spaced that each arch carries equal load. Each arch consists of a curved compression member, WT 18 by 150 with a 1-1/8 in. skin plate on the upstream side, and a tension tie girder, W 33 by 240 with the web in a horizontal plane.

During fabrication in 1962-63, numerous defects were found in the welds, and an extensive testing and repair program was undertaken. A large portion of the welding required repair before the gate was accepted.

In February of 1980, Portland District was informed by Walla Walla District that serious cracking had been found in the tension tie members in the navigation lock lift gate at the Ice Harbor project. Since the Ice Harbor gate is almost identical in design to the gate at John Day, a visual inspection was made and some ultrasonic testing was done on the John Day gate. No cracking was found; however, an annual inspection of the John Day gate was recommended and established.

On the annual inspection in 1982 serious cracks were found in several of the lower tension tie girders. One crack extended completely through one flange and through the web of the girder. An emergency lock outage was declared, and the cracks were repaired by welding. Also, cover plates were added to the downstream flanges of two of the tension girders, and gussets were added where the tension tie girders join the curved compression members. Since the repair, the gate has been inspected frequently and no cracking has been found.

A planar finite-element analysis was done on a typical tied arch in 1983 to determine if structural additions can be made to the gate that will lower stress levels in critical areas. Also in 1983 strain gages were mounted on

some of the lower arches to measure stresses with the gate in operation. The finite-element analysis indicated that the gussets which were added are effective in reducing stress concentration in the area where some of the cracks originated. Some of the strain gages indicated stresses agreeing with those calculated, while others were inconclusive. Future stress measurement using strain gages has been recommended.

The exact cause of the cracking is not known, however, the following certainly contributed to it: stress risers in highly stressed areas, vibration of the gate in operation, heavy ice loads, low temperature.

In addition to finding solutions to the problems on this gate: analysis and testing are of value for future designs of large welded steel structures.

For additional information contact William Wheeler, NPPEN-DB-SA, Comm: 503/221-6907; FTS: 423-6907.

## CONCRETE FLOATING BREAKWATERS

George G. England, P.E.  
Structural Design Section  
Seattle District

Predicting the wave forces acting on floating structures, understanding internal stress distributions, determining the optimum construction techniques, and providing an economical means of anchoring, connecting, and fendering the breakwater present many design problems. To provide guidance in solving these problems, Seattle District, in cooperation with OCE, Waterways Experiment Station, Coastal Engineering Research Center, and the University of Washington, conducted a prototype floating breakwater test program. To date the multitude of data are still being analyzed to evaluate conclusions and design recommendations.

Since implementation of the prototype breakwater program, two projects using floating breakwaters were designed and constructed by Seattle District. These projects were a 600-ft-long breakwater for East Bay Marina in Olympia, Wash., and a 1,600-ft-long breakwater installed at Friday Harbor, Wash. As information was gathered during the test program, it was used to benefit the design of the East Bay and Friday Harbor projects.

Cross-sectional changes to the float and broadening of construction specifications gave contractors flexibility in materials used to more easily obtain required flotation. These changes also improved constructibility of the concrete floats. Options were given for breakwater fabrication methods to increase numbers of possible bidders. Anchor systems performed extremely well for the three breakwater types in their locations. Details of East Bay's connector system were changed to increase reliability and decrease maintenance costs. Flexible connectors were removed at Friday Harbor and fenders were used to allow breakwaters to act independently of each other with some minor anchor system modifications.

For additional information contact George G. England, P.E., NPSEN-DB-ST, Comm: 206/764-3792; FTS: 399-3792.

FISHERMAN'S WHARF BREAKWATER STRUCTURAL  
DESIGN IN SAN FRANCISCO, CALIFORNIA

Gary W. Sjelin, P.E.  
Structural Engineer  
Los Angeles District

Fisherman's Wharf is the historic center of commercial fishing activity in northern California. The harbor area is exposed to wave action from the open ocean as well as from within San Francisco Bay, and is the most exposed small craft harbor in the area. As a consequence of continual damage to boats and difficulties in off-loading fishhatches during adverse weather, much of the fishing fleet have left for more protected harbors. This has jeopardized the viability of tourist industries in the area, which is the second largest tourist attraction in California. The breakwaters will provide protection for the return of the fishing fleet as well as for a Federally owned historic fleet.

The structural design of the Fisherman's Wharf Breakwater was a very unique and challenging task because of its unusual loading conditions such as 16 ft standing wave with maximum water depth of 75 ft. Also, the structure had to be designed to survive in an area of high seismicity.

The breakwaters consist of two basic types of design. One is a continuous wall with solid concrete sheet piles, and the others are 26-ft long segmented pile walls with 6-ft clear openings between segments to permit water circulation. The solid type will be approximately 1,500 ft in length while the east and west segmented breakwater will be 150 and 260 ft in length, respectively.

Both types of breakwaters will be constructed with interlocking precast-prestressed concrete sheet piles. In addition, precast-prestressed concrete batter piles at specified spacing will be provided on both sides of the sheet piles except the western end of the solid continuous type and the east segmented breakwater where batter piles will be installed on one side only.

A reinforced concrete pile cap with a width of 10 ft and a thickness of 4 ft will be provided on the top of all the pile walls except the west segmented breakwater where the width of the pile cap will be reduced to 7 ft.

The computer programs which were utilized in the design and analysis of the project are:

a. Design and analysis of sheet-pile walls by classical methods.  
(CSHTWAL).

b. Frame and dynamic analysis by GTSTRU DL with soil being modeled by linear springs.

Breakwater Design Loading Conditions.

a. Wave Forces: Standing Wave Height of 16.0 ft was considered as seaside waves and combined with variable lee waves (wharf side) to produce the most critical loading condition.

b. Scouring at mudline: The effect of a 10-ft scour on bayward side was combined with the wave forces above.

c. Seismic Load: The breakwaters were analyzed by assuming seismic loading conditions with a minimum earthquake coefficient of 0.2G. Neither wave forces nor scouring were combined with the seismic load.

Government estimated first cost of this project is \$12,600,000.

For additional information contact Gary Sjelin, SPLED-D1, Comm:  
213/688-5538; FTS: 8-798-5538.

## MITER GATE REHABILITATION

George G. England, P.E.  
Structural Design Section  
Seattle District

Since miter gates are an essential part of navigation lock operation, rehabilitation has become crucial to maintaining gate integrity. Recently, the Seattle District has rehabilitated several miter gate sets at its Hiram M. Chittenden Locks, a navigational facility completed in 1915. Being the first major rehabilitation since installation of the gates, problems have occurred in predicting accurate quantities of structural repairs from in-position inspections. Other problems include paint systems, specifications, and inspection, as well as acceptable contractor quality control.

Using lessons learned from each rehabilitation, improvements in initial inspection methods provide for better predictability in structural repair quantities reducing costly contract modifications. Changes in available paint systems, tightening of paint specifications and inspections increase longevity of rehabilitation while burdening contractor quality control personnel with more responsibility.

For additional information contact George G. England, P.E., NPSEN-DB-ST, Comm: 206/764-3792; FTS: 399-3792.

REHABILITATION OF VERTICAL LIFT GATES  
EMSWORTH DAMS

Eugene A. Ardine  
Structural Engineer  
Pittsburgh District

Emsworth Locks and Dams are six miles below the confluence of the Allegheny and Monongahela Rivers in Pittsburgh. The structures have been operated and maintained since 1921. The locks consist of two adjacent chambers, a main chamber of 110 ft by 600 ft and an auxiliary chamber of 56 ft by 360 ft, located along the right bank of the river. The dams, one on each side of Neville Island, originally constructed as uncontrolled fixed crest structures were converted between 1935 and 1938 to controlled structures which now control the navigation pool with vertical lift gates. The vertical lift gates, eight in the main channel and five in the back channel, composed of two horizontal trusses, two vertical girders, seven diaphragms, an overflow plate, and end frames originally functioned in two different modes of operation. The modes of operation were a nonoverflow mode where the gate is raised to pass all the flow under the gate and an overflow mode where the flow is over the gate and the gate designed to carry this additional load.

The vertical lift gates were observed as being severely deteriorated in June 1971 and a load comparison for the operating modes indicated it prudent to cease overflow operation. The extent of corrosion was of such a critical nature on many diagonal truss members that emergency replacement was necessary. An evaluation and rehabilitation program was performed on a pilot gate from which alternate repair and/or replacement plans were formulated and costs compared. Rehabilitation of the gates as per the pilot gate was found to be the most cost effective and a rehabilitation program was initiated using hired labor.

As of fiscal year 1979, only 50 percent of the rehabilitation work was performed and the projected hired labor scheduled work program indicated little or no hope for improving work progress. A reevaluation of the gates' deterioration and rehabilitation costs was necessary in preparation of a Major Rehabilitation Feature Design Memorandum for Emsworth Locks and Dams in FY 1980. An independent condition survey and evaluation of rehabilitation and

replacement alternatives by an architect-engineer firm recommended the retention of the rehabilitation program as the most cost effective even though repairs would be more extensive.

Plans, bid item selection, and specifications prepared from recommendations by the architect-engineer with District structural engineer modifications to simplify details and minimize cost proved effective by having only a three percent increase in construction cost. Some of the problems encountered during the work and responsible for the cost increases were: deterioration being more extensive than anticipated; rehabilitation details not applicable in all cases; the dimensioning of rehabilitation work based on as-built dimensions; and the differences of interpretations of such terms as watertight, level, and straightness. To eliminate some of these problems on future rehabilitation projects, the following recommendations are offered: be liberal in estimating repairs; include surface preparation in detail in all new items which are connected to existing structures; be leery of giving referenced dimensions tying new work to old members; and make sure all terms which require other than normal industry standard tolerances are distinctly defined.

For additional information contact Eugene A. Ardine, ORPED-DM, Comm: 412/644-6881; FTS: 722-6881.

References:

Feature Design Memorandum, Emsworth Locks and Dams, June 1980.

Plans and Specifications Emsworth Locks and Dams Rehabilitation.

## SILICA FUME CONCRETE REPAIR OF KINZUA DAM STILLING BASIN

Anton Krysa  
Structural Engineer  
Pittsburgh District

The stilling basin at Kinzua Dam, a Flood Control Project on the Allegheny River in northwestern Pennsylvania, has experienced extreme abrasion-erosion damage since the structure had been put into service in 1967. The basin was originally repaired in 1973-74 using a steel fiber-reinforced concrete overlay. Deterioration continued to the extent that repairs were again necessary in 1983.

A laboratory program was undertaken to evaluate the abrasion-erosion resistance of several concrete mixtures proposed for the 1983 repairs. This program showed that high-strength concrete made with condensed silica fume and limestone aggregates would provide a suitable abrasion-erosion resistance at a reasonable price. Potential suppliers of condensed silica were required to conduct full-sided placements to demonstrate that this concrete could be made and placed outside the laboratory. Based upon these demonstrations and laboratory program, the repair concrete was specified with a compressive strength at 28 days of 12,500 psi as a means of obtaining the required abrasion-erosion resistance.

Over 2,000 cu yd of 9-3/4-in. slump concrete were placed using condensed silica fume delivered as a slurry that included water-reducing admixtures. The average 28-day compressive strength actually attained was over 13,000 psi. Diver inspections of the concrete after two years in service, including a period with a very large volume of debris in the stilling basin, has indicated that the condensed silica fume concrete is performing as intended, indicating only a small amount of deterioration.

More detailed information can be obtained in a report entitled "Placement of Silica Fume Concrete and Rehabilitation Work at Kinzua Dam." Limited copies are available from the author: Anton H. Krysa, ORPED-DM, Comm: 412/644-5454; FTS: 722-5453. A good article with a general discussion on concretes with silica fume can be found in "Silica Fume Concrete - Properties, Applications and Limitations." V. M. Malhofra and G. G. Carrette, Concrete International, May 1983.

SAVANNAH HARBOR TIDE GATES,  
STRUCTURAL PROBLEMS AND REPAIRS

John W. Hager, P.E.  
Structural Engineer  
Savannah District

The Savannah Harbor Tide Gates are unique in their method of operation, function, and structural features. The function of the gates is to allow the flood tide to come up the Savannah Back River and close on the Ebb Tide. Retaining the flood tide increases the velocity in the Front River, thus decreasing the sedimentation rate and the dredging cost in the navigation channel. Decreasing the flow in the Back River increases the sedimentation rate. Since the Back River is closer to the spoil area, dredging in this river is more economical than dredging in the Front River.

The structure is located 20 river miles from the Atlantic Ocean and is subject to hurricane winds and wave action plus a 10-ft tidal range. There are 14 gates, each 27 ft high by 40 ft wide. The gates pivot about a pivot beam 18 ft from the bottom of the gate. Nine ft of the gate projects above the pivot beam where a box beam is attached. A hydraulic cylinder is attached to the center of the box beam where it reacts against the gate to force it open. Originally, the gates were to be powered open and closed throughout the tidal cycle. Because of structural problems, they were modified to freely swing with the tidal fluctuations of the river.

Several problems developed with the gates during the initial operation. When the structure was first flooded the tidal forces reacted on the gates to force them open, causing problems with the hydraulic pressure pipes and valves. The gates also trapped air under them, causing them to float on the flood tide and slam shut on the ebb tide.

When the gates slammed shut, large impact forces occurred causing the pivot pins to break. Investigations revealed improper welding and decreasing their impact strength. The gates were also forced closed against the reaction of the hydraulic cylinder, causing the pivot beam to buckle. The buckling strength of the pivot beam was reduced because the gates were also twisted during closure. These beams were later straightened and beefed up.

Because of possible hurricane winds the gates could be subject to severe wave action. A dynamic study of the gates was done to determine how the

reaction of the hydraulic cylinder would be affected by the motion of the gate under wave forces. The wave analysis showed that the gates could handle a wave force of no more than 60 mph and must be modified so that they could be locked for winds in excess of 60 mph.

This presentation will include slides on the gates showing typical buckling problems experienced with the pivot beam, a 3-D computer model and analysis of the gates for wave forces, racking, and the wave dynamics. Welding problems with the high strength stainless steel pivot pins will be explained. Also included in the slides will be a brief description on how the pivot beams were straightened by heat.

For additional information contact John W. Hager, SASSEN-DS, Comm: 912/944-5570; FTS: 248-5570.

DEMONSTRATIONS AND TRAINING SESSIONS

## CASE Introductory Session Descriptions

Monday, 24 June 1985  
2:45 p.m. - 4:40 p.m.

### Session 1

Miter Gate Analysis and Design. Program CMITER runs in time sharing to investigate or design a horizontally framed miter gate with as many as 50 girders when considering multiple load cases in accordance with EM 1110-2-2703, "Lock Gates and Operating Equipment." Data may be entered interactively or from a data file and may be edited on-line and saved for future runs. Investigation includes a complete stress analysis; design leads to a complete description of a gate.

U-Frame Lock Analysis. A program will be introduced which analyzes a lock monolith selected by the user from a menu of standardized types arranged to be representative of the most commonly used monolith geometric types. Soil and water loads and standard applied loads require only an overall description. Other loads may be described as desired. Foundation may be on earth or piles. Stability is determined by any of several methods as selected by the user. Stress analysis is by a beam/column method that yields forces and moments that are then used for design or investigation of concrete and steel.

Building Systems Analysis and Design. Three computer programs (CFRAME, CTABS80, and GTSTRU DL) that are useful for the analysis and design of buildings will be discussed. CFRAME is a general-purpose computer program for the analysis of small or medium plane frame structures subjected to static loadings. CTABS80 is a program which uses the special analytic properties common to building systems for linear 3-D structural analysis of multi-story frame and shear-wall buildings subjected to static or dynamic loadings. GTSTRU DL is a large general-purpose program typically used to analyze and/or design complex building framing schemes subjected to static and dynamic loadings. Each program strengths and weaknesses will be discussed briefly along with future plans of the Building System Task Group.

Finite-Element Method of Analysis. The use of the finite-element method (FEM) is becoming common within the Corps of Engineers. The Corps uses the FEM for solving a variety of complex structures with several load cases.

Because of the differences in using the method, the CASE Finite-Element Task Group has taken two common structures to be solved by using the FEM. The purpose of the studies is to provide guidance for using the FEM to aid in designing this particular structure. The guidance provides information on mesh density, effects of boundary conditions and proper methods for representing loads. The first two types of structures chosen were a gravity concrete non-overflow dam monolith (R. B. Russell) and a floor slab used in powerhouses.

The studies are of interest to any engineer doing FEM analysis in the Corps. The procedures from these studies can be extended to many other types of structures. Studies of this type always present information of interest to both the novice and experienced user of FEM.

## Session 2

Pile Foundations Analysis. Pile Group Analysis (CPGA) is a general-purpose program for determining structure deflection, pile head deflection, and pile head forces for 3-D pile groups. CPGA utilizes the stiffness method of pile group analysis for a rigid pile cap. Piles may be free standing and/or battered and the soil may be represented by a constant or linearly increasing horizontal subgrade modulus. The program has pile layout geometry generation capabilities. A number of auxiliary programs (pile interference check, pile stiffness generator, pile group optimization layout, plotting capabilities, etc.) to go with CPGA will also be discussed.

U-Frame Basins and Channels Analysis. This program has two user guides, one for basins and one for channels. Basins analysis follows EM 1110-2-2400, "Structural Design of Spillway and Outlet Works," 1956. Basins may have up to two intermediate walls, arranged symmetrically. Channel analysis generally follows EM 1110-2-2400. Channels may have one intermediate wall, located as desired. Analysis includes foundation stability pressures and internal stresses. Design procedures include determining slab and wall thicknesses and the size of required reinforcing.

Three-Dimensional Stability Analysis. The Three-Dimensional Stability Analysis/Design (3DSAD) program has been in use for some time now by various Corps offices. An overview of this program with an emphasis on new developments will be presented. For instance, a Design Memorandum (DM) plate capability has been added to CDAMS, and a new Gravity Lock Module is being

developed. Geometry and loads for typical chamber wall monoliths will be presented.

The new general-geometry capabilities, such as bicubic patches, axisymmetric blocks, rotation of objects in space, duplication of objects, and "clipping" by an arbitrary plane, will also be discussed. The last item to be described is the new Free-Body Module which uses the general clipping capability to form a new free body and then compute forces and moments about a specified point.

Sliding Stability Analysis. The Office of Chief of Engineers of the US Army released a new ETL on Sliding Stability for Concrete Structures (ETL 1110-2-256) back in June of 1981. This ETL contains new procedures for assessing the sliding stability of gravity dams and other concrete structures. An engineering study was undertaken by the CASE project to develop a computer program, SLIDE, to implement this new ETL. The basic procedures outlined in the ETL have extended for the computer program to provide capabilities for irregular soil surfaces, irregular soil boundaries, seepage pressures, uplift forces, single- and multiple-plane options for the failure surfaces. Input data may be supplied interactively from the terminal or from a predefined data file. The user can display the results on the terminal or store in an output file. The output contains the input data used in analysis, the factor of safety, and a summary of failure angles and forces acting on the wedges making the failure surface.

## Training Session Descriptions

### Session 1

Soil-Structure Interaction (SSI) Effects and Analysis Techniques. The status of the work of the Soil-Structure Interaction (SSI) research project. The work consists of evaluating simplified analytical procedures for the analysis of hydraulic structures using SSI principles. To date, only simplified procedures for plates/beam resting on a linearly elastic foundation have been examined. A common problem of a 100-ft long beam with different depths and three load cases will be used to evaluate the results of four SSI procedures. This includes extension of this work to structures with walls, and the anticipated end products of this work.

Retaining and Flood Walls Revised Draft Manual. An overview of the revised Retaining and Flood Walls draft engineer manual. The draft manual addresses the recommendations contained in the field comments on the draft of the manual distributed for review in FY 1984. Example problems are included to demonstrate some of the new guidance.

Design Procedure for Determining ANSI A58-1 Wind Loads for Buildings. A simplified design procedure to determine wind loads which comply with ANSI A58.1 requirements. Wind pressure tables and design notes are furnished for the main force resisting systems, walls, roof components, and cladding. Application of the procedure in the form of example problems is included.

### Session 2

Simplified Design Equations for Strength Design of Hydraulic Structures. The equations for axial and flexural capacity in ETL 1110-2-265. These equations cannot be used directly for design, but are presented in a format which is easily used just to analyze existing structures. The Vicksburg District has developed design equations for concrete members subjected to axial and flexural loading. These design equations are a direct, manual solution for the required reinforcement. This training session presents the theoretical background for these equations, and summarizes the steps used for design in accordance with ETL 1110-2-265.

Earthquake Analysis and Design of Concrete Gravity Dams, ETL 1110-2-303  
(In press). An overview of ETL 1110-2-303. This ETL is scheduled to be published by July 1985. The ETL contains guidance concerning: when a dynamic analysis is required; the sequence of the dynamic analysis; and the evaluation of the analysis results.

Ribbed Mat Slab Foundations Design. The ribbed mat foundation consists of a continuous reinforced concrete slab stiffened by reinforced concrete beams cast monolithically with the slab. These foundations are especially suited for use as shallow foundations on expansive soils. The training session includes design criteria presently used within the Corps, particularly in the Southwestern Division (SWD). The results of SWD studies being made to define the major parameters which influence the structural design are furnished.

## Training Session Descriptions

### Session 3

Review of Structural Engineering Guide Specifications and Recent Changes. The specifications training session will inform participants of new specifications, recent revisions to specifications, planned revisions to specifications, and continuing effort to solicit, exchange, and incorporate updated and revised information into both civil and military guide specifications.

Stability Criteria for the Rehabilitation of Navigation Concrete Structures. An overview will be given of the drafted ETL titled as above. The purpose of this ETL is to provide a standard procedure and uniform stability criteria for the rehabilitation of existing navigation concrete structures. The discussion will include: the background of this ETL; the general procedure used in evaluating the existing condition and determining remedial measures, if needed; the requirements for deviation from current criteria; and the use of stressed and unstressed anchors to stabilize the existing structures.

Earthquake Analysis and Design of Intake Towers Draft Manual. An overview of the draft engineer manual, Earthquake Analysis and Design of Intake Towers, will be given, and example problems of simplified methods of seismic analysis and design of circular and rectangular intake towers will be discussed.

EXHIBITS

## EXHIBITS

## Salon A

<u>Title</u>	<u>Type of Exhibit</u>	<u>Office/Contact</u>	<u>Phone</u>
Repair, Evaluation, Maintenance, Rehabilitation (REMR) Research Program--Introduction	Video	WES/Bill McCleese	601/634-2512 FTS 542-2512
Timber Repair of Roof Truss by Epoxy Injection	Video	St. Louis/Tom Quigley	314/263-5708 FTS 273-5708
Strength Design of Circular Concrete Conduits--Experimental	Video & display	WES/Steve Wright	601/634-2396 FTS 542-2396
Artificial Intelligence for Structural Evaluation	Computer demo.	CERL/Mike Lehmann	217/352-6511 Ext. 459 FTS 958-7459
Computer-Aided Structural Engineering (CASE) Project	Reports	WES/Paul Senter	601/634-3506 FTS 542-3506
Fixity of Members Embedded in Concrete Study	Report	St. Louis/John Jaeger	314/263-5693 FTS 273-5693
P-Version Finite-Element Program	Handout	St. Louis/John Jaeger	314/263-5693 FTS 273-5693
Analysis of Outlet Manifold, L&D 26R, Using GTSTRUDL H-Version Finite Elements with Independent Check Using FIESTA P-Version Finite Elements	Display & handout	St. Louis/Tom Leicht	314/263-5693 FTS 273-5693
Historic Steel Truss Bridge Floated Up with the New R. B. Russell Lake	Display	Savannah/John Hager	912/944-5570 FTS 248-5570
Shop Prestressing of Miter Gate Diagonals	Display	Pittsburgh/ John Menniti	412/644-5453 FTS 722-5453
Technique Used to Mine a Tunnel through John Day Powerhouse	Display	Portland/Dick Sinclair	503/221-6908 FTS 423-6908
Training Opportunities	Handout	OCE/Lucian Guthrie	202/272-8673 FTS 272-8673

# BREAKS



ATTENDEES

US ARMY CORPS OF ENGINEERS

OFFICE, CHIEF OF ENGINEERS

Dressler, Don  
Gibson, George  
Guthrie, Lucian  
Lee, M. K.  
McCormick, William N., Jr.  
Paavola, Ivar  
Smith, Robert  
Roper, William

EUROPEAN DIVISION

Mitrakos, Peter D.

HUNTSVILLE DIVISION

Lahoud, Paul  
Lein, Ron

LOWER MISSISSIPPI VALLEY DIVISION

Agostinelli, Victor M.  
Dubuissou, Roland J.

Memphis District

Kamin, Israel Sol

New Orleans District

Baumpy, Walter  
Hassenboehler, Thomas  
Romero, Jorge A.  
Schulz, Alan D.

St. Louis District

Naeger, John E.  
Flauaus, Richard J.  
Hoell, Roger J.  
Holt, Robert L.  
Jaeger, John  
Leicht, Thomas  
Mudd, Thomas J.  
Parks, Roy  
Quigley, Tom  
Ruf, Tom

Vicksburg District

Crow, Joe M., Jr.  
Dennis, Arvis R.  
Hamby, Clifton C.  
Perry, Allen L.

MIDDLE EAST DIVISION

Werner, A. O.

MISSOURI RIVER DIVISION

Johnson, John L.  
Churchill, William D.  
Staab, Ervell A.

Kansas City District

Bircher, Byron E.  
Price, Kirk M.  
Wright, Tom

Omaha District

Gaube, William  
Kelley, Bob

NORTH ATLANTIC DIVISION

Anastos, Stacey C.

Baltimore District

Simmons, James W.  
Veskimets, Enn

Philadelphia District

Capuzzi, Angelo M.  
Rambo, Augustus T.

NORTH CENTRAL DIVISION

Jacobazzi, Joe

Buffalo District

Lewandowski, Frank  
Ptak, Jerry

Rock Island District

Bigham, Jim  
Doak, Samuel S.  
Logsdon, Donald L.  
Lundberg, Denny  
Joers, Fred  
Johnson, Carl  
Wilson, Keith E.

St. Paul District

Cohen, Gerald L.  
Engstrom, Glenn  
Kliethermes, John C.  
Plump, John  
Spitzack, Charles

NEW ENGLAND DIVISION

Descoteaux, David R.

NORTH PACIFIC DIVISION

Fechner, Don  
Homan, Ed  
Ismail, Fahmi  
Joy, Jerry  
Laumand, Ken  
Madden, Dan  
Mroczkiewicz, Lou  
Raisanen, Dave  
Bickley, Chuck  
Ross, Dave  
Setvin, Mel  
Strom, Ralph  
Wisner, Douglas

Alaska District

Breeding, Paul  
Lam, Peter  
Leeak, Joseph B.  
Nott, Jim

Portland District

Friedenwald, COL Robert L.  
Katin, LTC Jon D.  
Daugherty, Ed  
Flanagan, Robert R.  
Hanson, Neal  
Cowing, Richard

Portland District (Continued)

Chambers, Donald R.  
Crump, Michael A. P.  
Dewey, Raymond R.  
Flynn, Michael C.  
Illias, David J.  
Johnson, Joseph  
Konno, Seichi  
Maurseth, Jerome  
McCracken, Bruce  
O'Toole, Terry G.  
Schmidtke, Brian  
Sedey, Jeffery  
Shuee, Daniel S.  
Sinclair, Richard J.  
Swanson, Karl  
Wheeler, William J.  
Hopman, Dennis  
Grey, Fong J.  
O'Brien, Molly

Seattle District

England, George  
Johnson, Garrett  
Nelson, Mike  
Noyes, Paul

Walla Walla District

Curtis, Charles W  
Hollenbeck, Robert E.  
Mora, Oswald L.  
Summers, Mark S.  
Collison, Bruce G.  
Wilke, Norman W.

OHIO RIVER DIVISION

Gaddie, Thurman  
Showers, William

Huntington District

Luicano, Pedro J.  
Taylor, Robert E.

Louisville District

Cozine, Larry M.  
Keith, Joe M.  
McClellan, Byron K.

Louisville District (Continued)

Snowberger, Ralph B.  
Speaker, John J.  
Wesley, Allan G.  
Gray, Herman  
Gray, Bill M.  
Gunnels, James E.  
Hull, Ken  
McClellan, Gordon J.

Pittsburgh District

Ardine, Eugene A.  
Krysa, Anton  
Menniti, John P.  
Riley, Bruce C.

SOUTH ATLANTIC DIVISION

Anderson, Leland D.  
Lewis, James G.

Jacksonville District

Navidi, Ray

Mobile District

Kimberl, Richard T.  
Parrott, Daniel L.  
Felder, Bobby B.  
Jordan, James R.  
Kling, Charles W.  
Fultz, Thomas L.  
Tashbin, Davood

Savannah District

Cheung, W. T.  
Close, Gary  
Hager, John W.  
Joshi, Kivti  
McAltine, CAPT Mike

Wilmington District

Lanier, Elwood G.  
Griffith, Gregory M.  
Nelson, Mark H.  
Woolwine, John B.

SOUTH PACIFIC DIVISION

Bergner, Donald L.

Sacramento District

Dunn, John  
Neff, David  
Bellet, Dennis  
White, John  
Haavisto, Bob  
Kristof, Susan

Los Angeles District

Ford, Clifford W.  
Sjelin, Gary

SOUTHWESTERN DIVISION

Hartman, Joesph P.  
James, Bill  
Veselka, Raymond

Galveston District

Comits, Peter

Little Rock District

Hill, David  
Lofton, Edward  
Papageorge, A. J.

Tulsa District

Henson, George  
Kikugawa, R. T.

CONSTRUCTION ENGINEERING RESEARCH LAB

Kao, Anthony  
Kearney, Frank  
Lehmann, Mike  
Prendergast, Dr. James D  
Preparata, Pat

WATERWAYS EXPERIMENT STATION

Mosher, Reed  
Tracy, Fred  
Woodson, Stanley C.

WATERWAYS EXPERIMENT STATION

(Continued)

Merrill, Chris  
Senter, Paul  
Price, William A.  
Hall, Dr. Robert  
Radhakrishnan, Dr. N.  
Kiger, Dr. Sam A.  
Pace, Dr. Carl E.  
Chiarito, Vincent P.  
Wright, R. Stephen

FEDERAL ENERGY REGULATORY COMM.

Foster, Jerry L.  
Mahoney, Daniel J.

CONSULTANTS

Dawkins, Dr. W. P.  
Hays, Dr. C. O.  
Inan, Dr. Mehmet I.  
Gleason, Scott

SOIL CONSERVATION SERVICE

Alling, Edwin S.  
Monville, Paul J.  
Saele, Leland M.  
Thackeray, David D.

TENNESSEE VALLEY AUTHORITY

Buttrey, Harold C.  
Suarez, Timothy M.  
Todd, Ernest

US BUREAU OF RECLAMATION

Dollar, David

CONFERENCE EVALUATION

STRUCTURAL ENGINEERING CORPS-WIDE CONFERENCE

24-28 June 1985

EVALUATION QUESTIONNAIRE

Name (optional): \_\_\_\_\_

Office: \_\_\_\_\_

Supervisor? YES (35) NO (61) 6 no response

If yes to above, FIRST LINE? (24) MIDDLE MANAGEMENT? (10)  
1 GS-12, Project Design Coordinator

A. ABOUT THIS CONFERENCE

1. How would you rate this conference overall?

Excellent (65) Good (32) Satisfactory (3) Unsa tory (0)  
2 no response

2. How did you like the format for this conference?

Excellent (57) Good (44) Satisfactory (1) Unsa tory (0)

If unsatisfactory, please explain why and make sugges , for  
improvement:

\_\_\_\_\_  
\_\_\_\_\_  
\_\_\_\_\_

3. How would you rate the organization of this conference?

Excellent (81) Good (20) Satisfactory (1) Unsatisfactory (0)

4. Did you think that the presentations made were well prepared and  
delivered?

All of them (13) Most of them (85) Few of them (2) None of them (0)  
2 written comments

Please list any suggestions for improvement.

a. \_\_\_\_\_  
b. \_\_\_\_\_  
c. \_\_\_\_\_

5. Were the visual aids (slides, viewgraphs, etc.) used by speakers satisfactory?

All of them (13) Most of them (87) Few of them (0) None of them (0)  
1 no response, 1 written comment

Please list any suggestions for improvement.

- a. \_\_\_\_\_  
b. \_\_\_\_\_  
c. \_\_\_\_\_

6. Were the CASE Program introductory sessions useful? YES (93) NO (5)  
1 N/A, 3 no response

7. How would you rate the CASE introductory sessions?

Excellent (19) Good (57) Satisfactory (21) Unsatisfactory (2)  
1 N/A, 2 no response

Please list any suggestions for improvement.

- a. \_\_\_\_\_  
b. \_\_\_\_\_  
c. \_\_\_\_\_

8. Were the training sessions Tuesday and Thursday afternoons useful?

YES (91) NO (1) 3 marked both, 5 no response, 2 written comments

9. How would you rate the training sessions?

Excellent (21) Good (57) Satisfactory (19) Unsatisfactory (1)  
3 no response, 1 marked good and unsatisfactory

Please list any suggestions for improvement.

- a. \_\_\_\_\_  
b. \_\_\_\_\_  
c. \_\_\_\_\_

10. Were the exhibits useful? YES (95) NO (2) 5 no response

Please list any suggestions for improvement.

- a. \_\_\_\_\_  
b. \_\_\_\_\_  
c. \_\_\_\_\_

11. Was the field trip interesting and useful? YES (93) NO (1)  
2 N/A, 5 no response, 1 both (yes & no)

12. Was the field trip organized well? YES (96) NO (2)  
2 N/A, 1 no response, 1 written comment

If no, please indicate what was wrong.

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13. Were the local and evening arrangements satisfactory?

Hotel rooms	Yes <u>(90)</u>	No <u>(2)</u>	2 N/A, 8 no response
Conference rooms	Yes <u>(93)</u>	No <u>(1)</u>	8 no response
Ice breaker	Yes <u>(74)</u>	No <u>(11)</u>	3 N/A, 14 no response
Wednesday lunch and dinner	Yes <u>(89)</u>	No <u>(2)</u>	2 N/A, 5 no response, 4 written comments
Spouse activities	Yes <u>(40)</u>	No <u>(2)</u>	17 N/A, 43 no response

If your answer is no, give your reasons.

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14. List 3 topics which were the most useful to you at this conference:

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B. FUTURE CONFERENCES

1. Do you think conferences of this type are helpful to disseminate and share information? YES (99) NO (1) 2 no response

List any specific benefits to you.

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2. List any presentations you want to see updated at the next conference(s):

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3. OCE sponsors one- or two-week short courses every year at WES through the Huntsville Division Training Office. Examples of these courses are Finite-Element Method, Computer-Aided 3-D Stability Analysis, Basic STRUDL, Design Criteria, etc. List 3 topics in which short courses could help your office most in FY87 and FY88.

FY 87

1. 

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2. 

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3. 

---

FY 88

1. 

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2. 

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3. 

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4. Where do you think the next structural engineering Corps-wide conference should be held and why?

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## DETAILED RESPONSES TO QUESTIONS

### Question 2, Section A

1. As a speaker I would have liked to have seen an objective evaluation of my presentation to gain insight where my presentation could have been improved. This evaluation could be by a panel or by the session chairman.
2. Keep civil works and military work in dual sessions so we have a choice of which session to attend. Audience should be in a room when session starts.
3. Some presentations by others (TVA, etc.).
4. I have difficulty sitting most of the day listening to people talk. I do like split sessions which allow moving back and forth.
5. The training sessions were not very helpful - eliminate them and go to a 4 day conference.
6. I'm surprised that there was no input from CRREL in this conference.
7. Very professionally run conference - some confusion at first as to where (room) specific concurrent sessions were held - perhaps a brief announcement would have sufficed.
8. The training sessions were an excellent idea. However, the time allocated was too short to make them effective. Suggest lengthening these sessions by eliminating some of the presentations.
9. Would have liked to attend both 9A and B, 18A and B, 22A and B. I can only suggest a little less on locks to spread remainder out.
10. Low level of technical presentation.
11. The opportunity to attend all lectures/presentations would be nice. If both concurrent seminars interest someone, it would be nice to have it at another time as well.
12. Conference topics: Too much time was spent on finite-element method projects - most work does not involve this complex analysis and the cost would not be justified to use it.
13. The concurrent sessions are frustrating because I always feel that I am missing 50 percent of some fascinating subjects.

### Question 4, Section A

1. Speakers should be told to prepare better visual aids material.
2. Too many photographs, not enough technical contents.

Question 4, Section A (Continued)

3. The general and concurrent sessions had well-prepared presentations. However, the training sessions seemed ill prepared. The speakers were either too detailed or too simplified with their presentation.
4. Require slides for sessions except for CASE and Training sessions (which are rather informal).
5. Any improvements in acoustics, reductions in distractions (noise), very worthwhile.
6. Presenters should arrive a few hours early and receive some guidance in use of visual aids/microphone, etc.
7. Several presentations were too technical to be adsorbed at a conference. The technical part could be covered in handouts.
8. Every presentation shall be provided with a write-up for better understanding of the presentation.
9. Videotape presentations should be considered since it can be polished into an excellent presentation and length of presentation can be controlled. Videotape also allows action.
10. Speak loud and clear. (3)
11. Speaker should repeat all questions asked. (2)
12. Engineers are not speakers!! Delivery from a text is uniformly deadly; form notes usually better; ad lib (those who did) best of all. (Of course, the freewheeling speakers usually were teachers, such as Radha.)
13. More guidance to people making presentations or preparation of slides and materials for easy viewing.
14. Package all notes and handouts into 1 handout.
15. Slides worked better than overhead projector.
16. Don't get involved in technical numbers. Show thought process and results. (2)
17. Consider more use of video rather than slide presentation.
18. Avoid step-by-step design, etc.
19. Presentations should identify the problem, outline the remedial action, and state lessons learned, without boring everyone with details.
20. Times of break or beginning of sessions begin at half hour increments, i.e., 10:00, 10:30, 11:00, etc.

Question 4, Section A (Continued)

21. Greater variety in presentation media - too many slide shows.
22. Need to adhere closer to handout on giving presentations.
23. Visual Aids (slides) - make sure they are readable. (5)
24. Possibly having a slide review committee as ACI does and notify this to presenters well in advance of conference so that there will be more effort made to ensure all slides are of good quality.
25. Have a better sound system so the speaker does not have to ask "Can you hear me in the back?"
26. Place the screen higher (if possible).
27. Have microphone stations for questions to be asked after each presentation.
28. Control feedback of microphones.
29. Do not read the presentations. (2)
30. Very good choice of topics - diversified and useful.
31. You (L. Guthrie) tried to stress importance of speakers repeating questions so the audience could hear. Next time the first session chairman who forgets this should be shot. This will be a very visible example for the rest of the speakers and chairmen.
32. Request speakers to use slides instead of viewgraphs unless they need to write on them. (2)
33. OCE staff should be more prepared when presenting ETL and Revised EM information.
34. Some speakers need to have more public speaking practice.
35. Not so much finite analysis, more on construction and/or engineering problems, so we can learn from others.
36. Develop guidelines for visual aids.
37. Handouts for support information.
38. Better summarization techniques.
39. Better preparation for graphics.
40. Suggest that each presentation be made to a "neutral" audience before the conference to work out bugs in presentation.

Question 4, Section A (Continued)

41. Have people practice on their speeches more.
42. All speakers should have worked on the project they are talking about.
43. Are copies of the presentations (not just abstracts) submitted to OCE for review prior to the conference? This may be a good idea, allowing for an evaluation of the entire subject matter.
44. Close each presentation with recommendations and/or lessons learned.
45. Videotape the presentations and make them available on a loan basis. These would be useful to those designing similar projects in the future to review the tapes.
46. The sound system did not always work properly (uneven coverage) - why not go to a wireless microphone?
47. Do not have 4 training sessions in a row.
48. Suggest meeting with speakers on Sunday night to go over microphone and/or audio-visual equipment usage.
49. Few speakers were not familiar with their slides; they had to read directly from their notes.
50. Each speaker must check if the audience can hear.
51. Presentations could be a bit more "alive."
52. Consider requiring the written reports be supplied if subjects are presented in concurrent sessions.

Question 5, Section A

1. Do not use viewgraphs with handwritten material.
2. Slides from contract drawings are nearly impossible; those prepared by visual arts people - even cartoons - are preferable. Underexposed or shadowed slides are frustrating.
3. Lines on drawings should be heavy.
4. Slides should use larger letterings.--10
5. Do not make slides from contract drawings.
6. Typed visual aids should not be used.
7. Some slides were too light; suggest using bolder colors not pastels.
8. Some slides were difficult to see.--5

Question 5, Section A (Continued)

9. Slides and viewgraphs of actual drawings were usually poor.
10. Dark green or blue on black does not show up very well. Send slide preparation instructions as early as you can. Mine were done before I got the instructions.
11. Visual aids add significantly to a speaker's presentation. Not all speakers utilized this.
12. Not enough photos of actual construction of projects.
13. Less use of overhead projector.
14. Have a preview area with same size screen as will be used in presentation room. Advise speakers ahead of time (before conference) that size screen will be used.
15. Slides are vastly improved since first conference and generally are very good. Several speakers showed computations and too many tables. I am sure that adequate guidance had been given.
16. Better color contrasting.
17. Minimize use of design computations slides.
18. Make sure all slides are in focus.
19. Better graphics on some slides.
20. When a speaker uses an overhead for viewgraphs in a small room, he needs to sit down.--2
21. Some problems occurred with the audio system.
22. Numbers are hard to see.
23. Avoid details.--8
24. Use the pointer.
25. Some could have been simplified to illustrate point, but most were ok.
26. A few were difficult to read if sitting more than halfway back in the room.--2
27. Use more video or film format.
28. Use visual aids to provide an overall view and if more detail is required, make each view simple with only a few concepts per view.

Question 5, Section A (Continued)

29. Visual aids should be prepared specifically for the conference. Trying to use slides not originally taken for that purpose doesn't always work that well. Those slides made specifically for the conference were excellent.
30. Do not use old project drawings with too much detail.
31. Some had too much lettering which people tried to read instead of listening to presenters.
32. Some were too complex; others were hard to read.
33. Some slides, particularly from WES, were too busy and hard to read. Too many graphs and charts.
34. Eliminate all use of viewgraphs.
35. Some viewgraphs were too complicated.
36. More animation would be more attention getting.
37. Some slides too dark.
38. Many were outstanding.

Question 7, Section A

1. Reduce the number presented and make each session longer.
2. More stress on limitations and/or of parameters programs.
3. Better organization of material.
4. Less details and more general information.
5. CASE program attempting to do too much. Better to concentrate on fewer tasks, more timely completion of on-going work.
6. Stick to overview and new topics. Don't repeat things we have heard before.
7. SSI got far too technical.
8. Should concentrate on giving us the results.
9. They were good to show what was available but should not attempt to teach how to use the programs.
10. Make sessions more general - the audience should be people unfamiliar with the program and who do not want all the details, just an overview.

Question 7, Section A (Continued)

11. Do not use conference as an excuse for a "CASE" vacation.
12. Provide handout material of the CASE program documentation and perhaps an example of input data.
13. Questions by members of audience difficult to hear (not always repeated by speaker).
14. Increase the time.
15. End presentation with example problems.
16. Should have spent more time on presenting their capability and merit.
17. Suggest that a list of CASE programs be made available so that individuals can check those programs that they are interested in receiving.
18. Give overall program capability instead of details that only those who have used the program are familiar with.
19. Why not just hold a general session on CASE programs (expand Dr. Radha's presentation) outlining the status and general description of each program being developed.
20. Need more examples and visual aids.
21. Show more of the program capabilities - using examples is a good way - and less of the formula development going into the programs.
22. Sessions were rushed; allow more time.
23. More background about CASE.
24. Provide better overview of program capabilities.
25. Some presentations were too detailed for just an overview.
26. Less technical and more emphasis on an overview of the programs.
27. Final complete "simplified" schedule for all sessions would be an improvement. (May require a revised distribution at start of sessions.)
28. One morning presentation could be used to cover all these CASE programs to the extent practical at a conference of this type.
29. Change format; spread sessions over several days.
30. Need more lessons learned from using the programs such as when to use a particular program to suit a job. Complex program for simple job, vice versa.

Question 9, Section A

1. CASE sessions were well prepared but presenters of training sessions on Thursday did not seem to be prepared.
2. Insufficient detail was usually presented to really encourage thought and discussion.
3. Eliminating the "general criteria" type discussions such as "training."
4. Delete the (Review of Structural Engineering Spec. change) talk - all information was on the handout.
5. A few speakers needed to speak up a little more.
6. Some of these sessions could have covered the matter in much less time, considering the information obtained. Examples: Wind Load, Corrosion, ETL on stability of Renovated Navigation Structure.
7. I found the regular sessions more useful than the 40-minute training sessions; however, the concept has very good potential.
8. Need a bigger screen for viewgraphs in smaller rooms.
9. Some of the presentations were merely read.
10. Provide sessions with more time (have 2 sessions instead of 4).
11. Provide military design training sessions for Earthquake (TM5-809-10) with braced systems, shear walls, and rigid frames.
12. More visual aids.
13. Expand the time spent on each session.
14. Not enough in-depth detail. The last conference in St. Paul was much better.
15. Add session for TM5-809-10 (Seismic Manual), Masonry Design, and Building Criteria.
16. Since our structures are experiencing corrosion and major rehabilitation, a training session with someone from CERC which would point out repair techniques, welding of stainless steels, cathodic protection, etc.
17. Could OCE promulgate, say, an ETL publicizing the simplified approaches strength design of hydraulic structures? ANSI wind loads? Could something similar be developed for the seismic TM5-809-10?
18. Extremely technical sessions hard to follow.
19. Suggest handout with any computations/methods shown.

Question 9, Section A (Continued)

20. Really need to make these sessions worthwhile, the time should be expanded to include specific examples of the information presented.
21. They need to "teach" something. Most did not have a well stated or evident training objective.
22. Include masonry design and other military problems.
23. Soil-Structure Interaction - too detailed, no useful information.
24. Sessions were too long.
25. Some presentations not clear, not well organized.
26. Follow-up on Stability Analysis, Gravity Dams, and Intake Towers.
27. Some training sessions had no purpose, no conclusion, recommendation, etc.
28. Session on ANSI wind loads needs improvement.
29. Some sessions were too technical for this type of forum (SSI in particular).
30. One or two sessions were too theoretical with no definite conclusions. Talks should be kept general with requests for specifics handled later.
31. Session on wind loading (especially second half) wasn't useful to me. Providing an example could be helpful. Sessions on Stability of existing structures and guide specifications had little content.
32. Each session should have prepared handouts.
33. Do not try to cover topics on which there is no new news.
34. Some sessions were very interesting and useful - others had no useful information or anything to say.
35. I do not find the majority of the training sessions valuable; I was looking forward to them but the depth and "intensity" of information presented was not up to the standards of the concurrent sessions.
36. Need military representation for guide specifications.

Question 10, Section A

1. Have an exhibit showing structural software for personal computers.
2. Group into subject displays.

Question 10, Section A (Continued)

3. Let people know they can provide exhibits instead of talks, so they can prepare an exhibit.
4. More interest could be generated if more P.R. was used.
5. More exhibits would have been good, or changing them every few days.
6. Give all districts/divisions opportunity to participate.
7. Encourage more participation in exhibit area.
8. Videotape some or all the presentations which could be loaned to district and division offices.
9. Have more exhibits.--4
10. Provide some slack time during day for viewing.
11. Now that it's too late, it's unfortunate that no vendors were invited to set up some micro-CAD systems out there, for example.
12. Advertise the contents of the exhibits at one of the general sessions.
13. Scheduling or noting when the exhibits can be set up at the earliest time.
14. Larger and more photos.
15. It would be helpful if at a specific time all the exhibits were "manned."
16. More of them from more districts would have been instructive.
17. No time to view.
18. Display draft manuals and guide specifications under revision. Include in program, highlighting significant changes. Allow for input at this meeting.

Question 12, Section A

1. Perhaps district personnel who worked on the design and construction could have been stationed at each location to describe some of the considerations and/or difficulties encountered.
2. Too much attention to biology, none to structural aspects.
3. Most spouses not interested in technical part of field trip. Suggest separate activity for spouses. Then meet for social activity.

Question 12, Section A (Continued)

4. I believe that the tour guide was unnecessary in that we did not need a high school student to show us around. It would have been fine to let us walk around on our own.
5. Too long at lock - tour guides should have been coached more - would have liked more time at the second powerhouse.
6. Needed a better plan for distributing box lunches.
7. Needed informed district personnel to conduct tour.
8. Less time at Bonneville powerhouse and less discussion of fish screens.
9. Could have had more free time during tours.
10. Would have liked to view the fish hatchery.
11. Provide tour guides that are more technically oriented.
12. Should have walked more at site. Too much time wasted getting on and off buses.
13. Need a guide to answer technical questions.
14. The day was too long. Why not use the entire day for the field trip, with more time for sight-seeing, and get back earlier? Supply a good lunch instead of a bad one and a good dinner.

Question 13, Section A

1. Ice Breaker - more seats should have been provided. Bar and food arrangement could be improved.
2. Wednesday lunch and dinner - many people are vegetarians or do not eat red meat. No option was considered for lunch or dinner.
3. Ice Breaker offered too little.
4. Ice Breaker - provide more bartenders and an easier access to food.
5. I received nothing for the \$8 icebreaker.
6. Availability of food was bad. Cost was high.
7. Thursday tour guides were not prepared and didn't know the area. Not enough time spent at Cannon Beach - most interesting stop of tour.
8. Wednesday lunch and dinner - lunch was not very well prepared, no drinks, packaging poor, items missing, and chips spoiled.
9. Conference room - feedback on microphone system was distracting.

Question 13, Section A (Continued)

10. Rooms were very cool.
11. Ice Breaker - snack lines laid out poorly and bar dismantled too quickly. Not a lot of nearby food choices on evening and weekends.
12. Wednesday lunch and dinner - drink shortage, lack of variety.
13. Wednesday lunch and dinner - I didn't care very much for the food. It certainly wasn't worth the cost.
14. Lunch was fine, but dinner was "so-so."
15. Ice Breaker did not have enough service.
16. Marriott Hotel didn't have a room for me and could not explain why the reservation was not made.
17. My wife did not attend spouse activities since they were all day events. This would make it difficult to accommodate small children on such long events.
18. Lunch wasn't very good.

Question 14, Section A

Topics that responders "thought were the most useful" and numbers of responders:

L&D 26R, Cofferdam Testing Program--1

Munitions Storage Magazines, Structural Failure and Evaluation of Steel Arch "Standard Design"--2

Crater Lake Hydroelectric Project--2

Status Report on the CASE project--5

Computer-aided Design and Drafting Studies--6

Micro-computers in Structural Engineering--1

R. B. Russell Forced Vibration Test--3

3-D Dynamic Analysis of Englebright Arch Dam--2

Simplified Load Factors for ETL 1110-2-265, Strength Design for Reinforced Concrete Hydraulic Structures--9

Military Project Design--2

Post and Panel Type Retaining Wall--5

Question 14, Section A (Continued)

Roller Compacted Concrete for Elk Creek Dam--4

Converting for In-House to A/E Contracting--2

"Fast Track" Design/Construction of the Rapid Deployment Joint Task Force  
HQ--1

The Bonneville Project and New Lock Design--1

L&D 26R First Stage Dam, Design and Construction Case History--6

Integrated Structural Engineering Support for the FEMA Key Worker Blast  
Shelter Program--2

Dynamic Soil-Structure Interaction Effects on and Reinforcement Details for  
Blast Shelter Program--3

Restoration of Building 3001, Tinker AFB--9

Problems with Long-Term Concrete Deflections in a Hot, Arid Climate--3

Thermal and Stress Analysis for Longitudinal Joints for Three Gorges Dam--2

Savings through Engineering on the Downstream Guidewall at L&D 26R--2

Underground Munitions Storage Facilities Study--1

Underground Munitions Storage Complex, Design--1

L&D 26R Lock, Design and Construction sequence--1

Dam Safety, A Comprehensive Approach--2

Structural Behavior of Miter Gates--2

Structural Behavior of Alternate Configurations of Miter Gates--2

Mud Mountain Dam Intake Tower Analysis--3

L&D 25 Guidewall Repairs and Stabilization--2

Corrosion Protection for Projects in Louisville District--1

Investigation and Repair of John Day Navigation Lock Downstream Lift Gate--3

Concrete Floating Breakwaters--3

Miter Gate Rehabilitation--1

Rehabilitation of Vertical Lift Gates--1

Question 14, Section A (Continued)

Silica Fume Concrete Repair of Kinzua Dam Stilling Basin--2

Illinois Waterway Lockport and Brandon Road Locks 1984 Rehabilitation--3

Miter Gate Analysis and Design--2

Finite-Element Analysis--2

Pile Foundation Analysis--3

3-D Stability Analysis--1

Sliding Stability Analysis--1

SSI Effects and Analysis Techniques--1

Retaining & Flood Walls Revised Draft Manual--5

Design Procedures for Determining ANSI Wind Load for Buildings--6

Simplified Design Equations for Strength Design of Hydraulic Structures--3

Earthquake Analysis and Design of Concrete Gravity Dams, ETL 1110-2-303--1

Stability Criteria for Rehabilitation of Navigation Concrete Structures--3

Earthquake Analysis and Design of Intake Towers Draft manual--1

Design Features of Flood Walls--3

Use of Finite-Element Methods by different offices--1

Concrete Reactivity Problems--2

Gravity Dam Seismic Design ETL--1

All L&D 26 Presentations--3

Strength Design of Concrete--1

Lockport Rehabilitation--1

Rehabilitation of Lock Walls and Gates--1

Structural and Arch Features of Flood Walls--1

Concrete Design--1

Rehabilitation Discussions--2

Miter Gate Presentation--1

Question 14, Section A (Continued)

Status of Draft ETL's, EM's, etc.

Engineers' Responsibility for Structural Integrity--1

Building Design and Construction Presentations--2

CASE Training Sessions--4

Miter Gate Studies--2

Military Building Presentations--1

Spirit Lake - Don Chambers--1

Keynote Address by William McCormick--4

CASE Programs--2

L&D Design--1

Practical Construction Techniques--1

Dynamic Analysis of Intake Towers--2

Dam Design--1

Stability of Structures--1

Finite-Element Topics in General--2

Military Design--1

Concurrent CASE Sessions--1

Presentations on Cofferdam Design--1

Presentations on Miter Gate Design--2

Miter Gate Lectures--1

Dynamic Analysis Lectures--1

Miter Gate Rehabilitation--1

Dynamic Analysis of Concrete Dams--1

Rehabilitation of Structures--2

Military Building Program--2

Foundation Problems--2

Question 14, Section A (Continued)

Designs Involving Soil-Structure Interaction--1

The Posttensioning Failures on the Old River Control Aux. Structure--7

Structural Failure of Steel Arch Munition Storage Magazine--1

Fisherman's Wharf Breakwater--1

Question 1, Section B

1. Discussion with others of common problems and solutions achieved. Exposure to other districts' specific site and environmental situations design and considerations given for unique solutions.
2. Provide pretty good state-of-the-art information.
3. Provide good ideas that could be used in other projects, etc.
4. Personal contact with division, OCE, and WES personnel. Found handout at back of room very useful and informative.
5. I have been with the Corps for 16 months so all the information is helpful. I feel a good percentage of those attending such a conference are 1-2 year engineers.
6. Learning of problems and solutions in other districts is useful.
7. Learn of various experiences of other engineers in the country.
8. Handouts furnished and contacts made.
9. Gain experience through the conference.
10. Get to know other structural engineers and therefore have future help.
11. Found many areas which need further research. Contacts with these offices will be valuable for further work.
12. Meeting with other structural engineers Corps-wide.
13. Seeing and hearing what other districts are doing.
14. Major benefit is that I have a list of projects and people to contact for information if similar projects and design problems occur at my district office.
15. When future problems arrive, I will know where to go for experienced help in the Corps.
16. Share experiences and knowledge.

Question 1, Section B (Continued)

17. Face to face exchange of information. Personal contact with other districts.
18. As stated in the question, to obtain up-to-date information in developments throughout the Corps, and to maintain contacts with other engineers who may be a future help on questions arising on future projects.
19. Structural design for military projects.
20. It helped me understand better why the Corps did not just use ACI with increased load factors for concrete design. It also helped me understand how to use the strength design method.
21. Also important to meet other Corps of Engineers structural engineers face to face and see how much other districts and divisions do business.
22. Establish references for similar problems.
23. I have never failed to take several ideas back to my district and applied these ideas to our projects.
24. Awareness of information available on specific structural subjects.
25. Gives a chance to understand the problems encountered in other districts on rehabilitation military construction, etc.
26. Sources of contact for future design problems.
27. Contact and idea exchange with other engineers; good for the psyche.
28. Meeting people whose work I have previously read or reviewed; getting in touch with the task group.
29. Learn about others' problems/solutions.
30. New design methods, applications.
31. Exchange of experience and philosophy.
32. Share information with others; get to know other engineers.
33. Became more aware of other districts activities, and their approach to resolving problems. Meeting other engineers.
34. Provides places to find specific structural applications for future designs.
35. New information, awareness, contacts, interchange of ideas and philosophies, and insight into criteria. After experiencing this conference, I feel PROUD that I am part of the CORPS.

Question 1, Section B (Continued)

36. Increase my awareness of who in the Corps has expertise or performed research in a particular project. I will know where to call for additional information, thus avoiding duplication of effort.
37. Seeing projects and design applications of other districts; general knowledge of structures under construction. (Civil and military works).
38. Confirmation of our approaches and solutions to problems by knowing that they have been used independently by other structural designers.
39. Inclusion of several military presentations has made the conference more representative of the work accomplished by the Corps than the previous conferences. This improved my understanding of how other divisions/districts design and manage military construction.
40. Overall knowledge of how various districts approach different projects and also the lessons learned type lectures are very helpful.
41. Gave an insight on how other Corps offices approach different problems.
42. Greatest benefit is sharing technology and research in areas of common interest. Every organization benefits. Enhances development of individuals as well as the profession.
43. Makes me aware of projects and problems similar to our projects and problems.
44. Too many to mention.
45. Sharing experiences with other districts, gaining insight from experienced engineers.
46. Better understanding of design concepts and problems.
47. Keep up to date and broaden my knowledge.
48. Problems and corrective measures used on posttensioned/anchors.
49. General exposure to variety of topics; several presentations on design features that I am currently involved in.
50. Good insight on current Corps philosophy by Mr. Bill McCormick, Wind load discussions, formal and informal. Seeing what the Corps is doing broadens thinking.
51. I believe the benefits derived by the dissemination of engineering information and personnel contacts will pay back the costs many times over by saving of future design time. We do not have to invent the wheel over and over again.
52. To see the activities and projects of the other Corps offices.

Question 1, Section B (Continued)

53. I would hesitate less to call on someone else from another district to draw on their experience and to get their advice.
54. Exposure to a broad range of concepts, and meeting many new engineers to contact for consultation in the future.
55. Gave time saving ideas and methods and new approaches to the solution of problems.
56. Presented different views of structural design within the Corps.
57. Soil Conservation Service has many of the same problems as the CE, but on a smaller scale - we can benefit from your experiences.
58. Obtained knowledge of work methods being used by others; construction techniques shown will be helpful in planning and estimating future work.
59. Establish contact for similar problems.
60. Obtain information not known or available.
61. General insight into Corps activities.
62. Obtain ideas for future designs.
63. Identify critical design parameters for special problems.
64. Guide specification information.
65. Contacts with other divisions and districts with others such as TVA. An opportunity to discuss mutual problems and solutions.
66. Learning of approaches to problems by other districts.
67. Learning about changes in criteria (i.e., EM's, ETL's, etc.).
68. Was able to meet people I had only had phone contact with before and discuss their views in greatest detail. Also, in answering questions throughout the week on my talk, I was better able to understand some of the questions I had myself.
69. Exposed to other peoples' problems and solutions.
70. Meet other designers.
71. Instill pride in Corps, its accomplishments, and its people.
72. Exchange of experiences with O&M problems.
73. Personal contacts.

Question 1, Section B (Continued)

74. Information on design work done by other districts.
75. Meeting people of the same discipline for future contact.
76. Learning the types of designs others are doing and the problems or successes they are having.
77. To better understand the work ongoing in other districts.
78. They would be more helpful if more on-line "production" engineers were in attendance at the expense of some supervisors.
79. I know a lot more about the use of visual aids.
80. Growth of knowledge of not every day topics.
81. Provides an opportunity of seeing what structural problems the districts have faced and how they solved them cost effectively.
82. Better understanding of work in other districts - plus our own district.
83. Sharing of problems/solutions for design, construction, and rehabilitation problems.
84. To find out how someone else approached a problem similar to one I face.

Question 2, Section B

Presentations that responders "want to see updated at the next conference" and numbers of responders:

Munitions Storage Magazines, Structural Failure and Evaluation of Steel Arch  
"Standard Design"--1

Crater Lake Hydroelectric Project--8

Status Report on the CASE project--8

Computer-aided Design and Drafting Studies--2

Tactical Equipment Shops, Best Design and Layout Study--1

Simplified Load Factors for ETL 1110-2-265, Strength Design for Reinforced  
Concrete Hydraulic Structures--5

Military Project Design--1

Roller Compacted Concrete for Elk Creek Dam--3

Converting for In-House to A/E Contracting--2

Question 2, Section B (Continued)

L&D 26 Construction--13

L&D 26R--2

Integrated Structural Engineering Support for the FEMA Key Worker Blast Shelter Program--1

Dynamic Soil-Structure Interaction Effects on and Reinforcements Details for Blast Shelter Design--1

Restoration of Building 3001, Tinker AFB--1

The Bonneville Lock and Dam project--5

Unique Factors Influencing the Design and Construction of the Ft. Campbell Hospital--1

Thermal and Stress Analysis for Longitudinal Joints for Three Gorges Dam--1

Underground Munitions Storage Facilities Study--1

Dam Safety, A Comprehensive Approach--1

Structural Behavior of Alternate Configurations of Miter Gates--2

Cerrillos Dam Outlet Works--3

Mud Mountain Dam Intake Tower Analysis--3

L&D 25 Guidewall Repairs and Stabilization--1

Corrosion Protection of Projects in Louisville District--1

Investigation and Repair of John Day Navigation Lock Downstream Lift Gate--1

Concrete Floating Breakwaters--2

Silica Fume Concrete Repair of Kinzua Dam Stilling Basin--1

Spirit Lake Outlet Tunnel--2

Miter Gate Analysis and Design--1

Sliding Stability Analysis--1

Soil-Structure Interaction Effects and Analysis Techniques--1

Retaining & Flood Walls Revised Draft Manual--2

Design Procedure for Determining ANSI Wind Load for Buildings--1

Question 2, Section B (Continued)

Earthquake Analysis and Design of Concrete Gravity Dams, ETL 1110-2-303--1  
Stability Criteria for Rehabilitation of Navigation Concrete Structures--1  
Earthquake Analysis and Design of Intake Towers Draft manual--2  
Soil-Structure Interaction - Results only--1  
Strength Design Criteria--1  
Contractor Quality Control--1  
Miter Gate Finite-Element Study--1  
Ultimate Strength Concrete Design--1  
Slides of Current Construction--1  
Update report on CADD usage--1  
Update manual of gravity dam design--1  
Vanderberg AFB construction program (space shuttle)--1  
CASE training session--1  
Gravity Dam & Intake Tower Seismic Design Manuals--1  
Military construction--1  
Lock Rehabilitation--1  
Serviceability of Concrete Repairs--1  
Electrical/Mechanical Replacement During Rehabilitation--1  
Strength Design of Concrete--1  
Presentation on sliding criteria--1  
Presentation on Stability of Intake Towers--1  
Bureau's work on Chinese Dams--1  
Stability of Existing Concrete Structures--1  
Other rehabilitation projects--2  
Progress on REMR Activities--1  
Earthquake Studies--1

Question 2, Section B (Continued)

Career Development and PE Registration--1

Design of Bonneville Second Lock--1

Construction of Fisherman's Wharf Breakwater--1

Observation of Munitions Bunkers at Sunflower, etc.--1

Question 3, Section B

Topics that responders "think short courses need to be conducted" in FY 87 and the number of responders:

Analysis

- 3-D stability analysis--1
- Building analysis--1
- Computer-aided 3-D stability analysis--8
- Dynamic Analysis on Intake Structures--1
- Dynamic analysis--1
- Earthquake analysis--3
- Miter Gate analysis--1
- Pile Foundations analysis--3
- Sliding Stability analysis--1

Design

- Building design--1
- Building system design--1
- Computer-aided design of buildings--1
- Concrete design--2
- Concrete channel design--1
- Design criteria--15
- Design and construction of breakwater structures--1
- Design of gravity dams--1
- Design of hardened facilities--1
- Design of intake towers--1
- Design of locks, spillways, and powerhouses--1
- Design of tunnels--1
- Earthquake design--1
- Masonry design for buildings--4
- Military design criteria--3
- Miter gate design--3
- Railroad bridge design--1
- Review of structural design criteria--1
- Security aspects of facility design--1
- Seismic design--6
- Seismic design (TM 5-809-10)--4
- Sheet pile design--1
- Steel design--1
- STRU DL as a design aid--1
- Ultimate strength concrete design--1
- Welding design--1

Question 3, Section B (Continued)

Application of CADD and lessons learned--1  
Bonneville Navigation Lock--1  
CASE programs available - usefulness and limitations--1  
Cerrillos--1  
CFRAME training--1  
Computer-aided drafting--1  
Concrete Technology--1  
Corrosion protection--2  
CSLIDE--1  
CTAB 80--4  
Elk Creek--1  
Finite-Element Methods--16  
GTSTRU DL--3  
Lock and Dam 26R--1  
Nuclear fallout shelter design--1  
PC software applications--1  
Project rehabilitation--1  
Rehabilitation of existing structures--1  
Retaining and flood walls (Workshop on new manuals)--1  
Statistics for civil engineering applications--1  
Structural dynamics--1  
Soil-Structure interaction--5  
Stability walls and other concrete structures--1  
SEAO Code (blue book) after 85 update--1  
Structural building systems--1  
STRU DL (basic)--15  
SHTWAL--1  
Use of all CASE programs--2  
U-FRAME--1  
More courses oriented to military projects--1

Topics that responders "think short courses need to be conducted" in FY 88 and the number of responders:

Analysis

Computer-aided analysis program--1  
Computer-aided 3-D stability analysis--1  
Finite-element method for stress analysis of concrete structures--1  
Miter gate analysis--1  
Plate analysis--1  
T-wall design and analysis--1

Design

3-D stability--1  
Computer-aided design of buildings--2  
Computer-aided design of hydraulic structure on flexible foundation--1  
Computer-aided design of U-FRAME with CBNTBM--1  
Concrete design--1  
Design criteria--5  
Design of hardened facilities--1  
Design of locks, spillways, and powerhouses--1

Question 3, Section B (Continued)

Flood walls--2  
Hydropower layout and design--1  
Masonry design--1  
Military design--1  
Miter gate design--1  
Retaining walls--3  
Review of structural design criteria--1  
Sector gate design--1  
Security aspects of facility design--1  
Seismic design--2  
Sheet pile design--1  
Steel design--1  
Welding design--2  
Wind design (ANSI)--1

Building Criteria--1  
CADD use and implementation--1  
Computer-aided drafting--1  
CTABS 80--1  
Databack management and spreadsheet programs--1  
Earthen dam--1  
Finite-element method--6  
How to use CASE programs--1  
Military construction--1  
Rehabilitation of existing structures--1  
Review of SEAOC Blue Book Update (1985) proposed publication--1  
Sheet-pile structures--2  
Sliding stability of concrete structures--1  
Soil-structure interaction--6  
Statistics for civil engineering applications--1  
Structural dynamics--1  
Structural Guide Specifications overview--1  
STRU DL (advanced)--2  
STRU DL (basic)--4

Question 4, Section B

Places that responders "would like the next conference to be held," number of responders, and reasons why:

1. Alaska--7

- a. Exciting and interesting environment which has presented us, and will continue to present us, with varied engineering challenges. A Corps conference held in this relatively new state can be interpreted as providing well deserved recognition to this state for meeting these challenges and for accomplishments there.
- b. Unique features and examples.

Question 4, Section B (Continued)

- c. Due to the area, and a chance to make them a part of a conference. Also it is a site of 2 major dams under construction.
  - d. Interesting civil and military projects.
- 2. Albuquerque--1
- 3. Atlanta--3
  - a. Richard Russell Dam
  - b. Majority of participants come from central and eastern United States.
- 4. Baltimore--1
- 5. Boston--6
  - a. Assuming that it will be the East Coast's turn, it is an area of engineering and historical interest.
  - b. Historical area.
  - c. To have more sessions on projects from eastern United States.
- 6. Dallas--3
  - a. More centrally located.
- 7. Florida--1
  - a. To see the space center.
- 8. Fort Worth--3
  - a. Climate and accessibility.
  - b. More centrally located.
  - c. Has a District office.
- 9. Hawaii--5
  - a. Climate and cheap transportation during summer seasons.
  - b. I've never been there.
  - c. Good convention locations.
  - d. Maybe the only place that can top Portland.

Question 4, Section B (Continued)

10. Hilton Head (North Carolina)--1

- a. It's beautiful and the coastline would be a nice setting.

11. Jacksonville--2

- a. I think these conferences should be moved around the country. Jacksonville is a logical place to hold the conference in the southeast.

12. Kansas City--2

- a. Located in central portion of United States.

13. Las Vegas--2

- a. More centrally located.
- b. Good hotels and conference rooms, which have been used successfully by ASCE, PCI, World of Concrete, etc. Good field trips to Lake Mead and Hoover Dam and Dept. of Energy test sites.

14. Los Angeles--4

- a. Has a District office.
- b. Varied projects.
- c. Good location, varied design examples.

15. Louisville--1

- a. Central location, interesting city.

16. Nashville--1

17. New England Division--7

- a. Conferences have been in the West and Midwest already.
- b. To have a chance to see and learn more about the coastal work that the Corps is involved in.
- c. Reduce travel costs for East Coast participants.
- d. Nice location and conferences have not been held in the East.
- e. Most training courses are at WES and HND; good to see other parts of the United States and so much to see if held at New York, or Washington, DC, or Boston.

Question 4, Section B (Continued)

18. New Orleans--7

- a. Central location, varied design examples.
- b. More central location.
- c. Facilities and technical capability.
- d. Excellent hotels with many entertainment opportunities.
- e. Qualified in all areas on consideration.

19. New York--1

20. North Atlantic Division--1

21. Omaha--2

- a. Interesting projects.
- b. Central United States location.

22. Pensacola--1

- a. I have an old girlfriend there.

23. Phoenix--2

- a. Warm climate.
- b. Excellent conference facilities and excellent visit possibilities to Roosevelt Dam, Stuart Mountain Dam, Horse-Shoe and Bartlett Dam.

24. Pittsburgh--1

- a. Facilities and technical capabilities.

25. Portland--1

- a. The Portland District is always short of travel money; we are very often cancelled out of classes for this reason; if the conference were not held in Portland, chances are that very few, if any, people would be able to attend.

26. Puerto Rico--3

- a. To see the Arch Dam.
- b. Since the excellent excursion rates would be obtained for travel. Also site of two major dams under construction.

Question 4, Section B (Continued)

- c. A good change in environment plus a good place to appreciate the dam construction being held there.
- 27. Sacramento--3
  - a. Chance to visit Corps' Warm Springs Dam
  - b. Proximity to Yosemite, Tahoe, Ocean, etc.
  - c. Varied projects.
- 28. San Francisco--6
  - a. Significant structural engineering accomplishments that can be observed.
  - b. Good location, chance to visit Corps' Warm Springs Dam.
  - c. Good convention location.
- 29. St. Louis--19
  - a. Centrally located in United States--3.
  - b. Lock & Dam 26--12.
  - c. Has a District office.
  - d. Facilities and technical capability.
  - e. Since we had 10 St. Louis personnel attend, it might be cheaper to bring the conference to St. Louis.
- 30. Washington, DC--5
  - a. Participants could see the role OCE plays in the Corps mission.
  - b. A lot of us, especially the newer engineers would like to see where and what OCE is all about.
- 31. Waterways Experiment Station (Vicksburg)--1
  - a. To find out more about what kind of research and development WES is involved in.
- 32. Watertown, N.Y.--1
  - a. Fort Drum

Question 4, Section B (Continued)

33. East Coast--2
  - a. Most conferences have been held in the West.
34. West Coast--1
35. An eastern district; we have had two in the West and one central (St. Paul), it would be nice to see how the easterners do things.
36. Northeastern part of the country. I would like to see what type of facilities the Corps operates and maintains in this area.
37. Opportunity should be given to various geographically different Corps districts to host the conference to provide the influence of local environmental conditions influencing design presentations to be introduced to the attendees.
38. I don't think it matters much where its held as long as the facilities are good. Probably should be where travel would be minimum for most people.
39. Another first class hotel like the Marriott.
40. Conferences should be rotated to all divisions.
41. Try a resort setting.
42. Any city which has unique qualities, such as beautiful country sides or outstanding food, so that the conference remains a work/visit type week.

General Comments

1. I would have appreciated a presentation by a local engineer about structural engineering accomplishments in Portland. There are so many impressive buildings and bridges that it is a shame to travel 3,000 miles to attend a conference without learning about the local accomplishments of our contemporaries.
2. Panel Discussion worked well - recommend to schedule again at next conference.
3. Why not have a speaker or two from the industry who have opinions about COE procedures and design criteria that we could learn from. (Suggestion - Sigmund Freeman, one of the writers of TM5-809-10).
4. More military projects should be included at the next conference.
5. Thanks for ordering excellent weather.

General Comments (Continued)

6. Portland District personnel did a super job in the arrangements, accommodation, and tours. Especially commendable were Seichi, Dave, Fong, and Mollie for the personal attention they gave me, my wife, and son.
7. Recommend that A/E Design Firms and University professors who do not work for the Corps be invited to make presentations at next conference.
8. The field trip was a tough job handled very well.
9. Many of the computer-aided problems reminded me of physics problems when I was in college. They would say "neglect friction." Many of these models seem to neglect some basic loading conditions, thus making them a bit unreliable.
10. Field trip was very enjoyable.
11. Field trip was very good, well run and coordinated.
12. After Monday, the sound system improved.
13. More time ought to be allowed for registration to have a way planned to process preregistration before the conference (payment too).
14. I would suggest conferences be held every 6 years instead of present 3 with division conference in between.
15. Papers should be submitted to ASCE.

END

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